

**HAM-75-7.85
RETAINING WALL K
PID NO. 77889
HAMILTON COUNTY, OHIO**

DRAFT STRUCTURE FOUNDATION EXPLORATION REPORT

Prepared For:
EMH&T

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Columbus, Ohio 43054**

Prepared By:

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Rii Project No. B-10-020

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June 15, 2012 (Revised August 8, 2013)

Mr. Edward D. Kagel, P.E.
Director of Transportation
EMH&T
5500 New Albany Road
Columbus, Ohio 43054

**Re: Draft Structure Foundation Exploration
HAM-75-7.85
Retaining Wall K
PID No. 77889
Rii Project No. B-10-020**

Dear Mr. Kagel:

Resource International, Inc. (Rii) is pleased to submit this DRAFT structure foundation exploration report for the referenced project. Engineering logs have been prepared and are attached to this report along with the results of laboratory testing. This report includes recommendations for the design and construction of proposed Retaining Wall K as part of the HAM-75-7.85 project. The proposed wall will be located south of the Paddock Road and I-75 interchange on the north side of Cincinnati, in Hamilton County, Ohio.

We sincerely appreciate the opportunity to be of service to you on this project. If you have any questions regarding the structure foundation exploration or this report, please contact us.

Sincerely,

RESOURCE INTERNATIONAL, INC.

Brian R. Trenner, P.E.
Project Engineer

Jonathan P. Sterenberg, P.E.
Director of Geotechnical Services

Enclosure: DRAFT Structure Foundation Exploration Report

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EXECUTIVE SUMMARY

Resource International, Inc. (Rii) has completed a DRAFT structure foundation exploration report for the design and construction of proposed Retaining Wall K as part of the HAM-75-7.85 project. It is understood that this wall will be connected to the rear abutment of the proposed HAM-561-7.01 Seymour Avenue over I-75 bridge structure at the south end of the wall alignment and will extend north along the west side of the proposed Ramp C. According to design details provided by EMH&T the rear abutment for the bridge structure is currently proposed to be a full height cast-in-place (CIP) wall type abutment with associated wingwalls extending to the north and south of the structure. To provide continuity, it is understood that a CIP wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall K. The total wall length is approximately 450 lineal feet.

Exploration and Findings

On October 3 and 4, 2011, a total of four (4) structural borings, designated as B-040-0-11 through B-043-0-11, were drilled to depths ranging from 25.0 to 50.0 feet below the ground surface at the locations illustrated on the boring plan presented in Appendix II of the full report.

All of the borings were drilled along the existing embankment adjacent to the I-75 southbound lanes and encountered 4.0 to 8.0 inches of topsoil at the existing ground surface, identified by the significant presence of organic matter and vegetation.

Beneath the surface material in borings B-040 and B-043, material identified as existing fill was encountered extending to a depth of 8.0 and 5.5 feet below the ground surface, respectively. The fill material consisted of dark brown and brown gravel and sand (ODOT A-1-b) and contained brick fragments.

Underlying the surface materials and fill material in borings B-040 and B-043, natural soils were encountered consisting of granular soils overlying cohesive soils. The granular soils were generally described as brown and brownish gray gravel, gravel and sand, gravel with sand and silt and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3). The cohesive soils were generally described as gray, brownish gray and brown silt and clay and silty clay (ODOT A-6a, A-6b) with lesser percentages of gravel and/or sand.

Bedrock was not encountered in any of the borings performed for this exploration.



Analyses and Recommendations

Design details of the proposed retaining wall were provided by the Rii design team. It is understood that the proposed retaining wall will be a reinforced concrete cast-in-place (CIP) wall type that will connect to the northwest wingwall of the rear abutment of the proposed HAM-561-0701 bridge carrying Seymour Avenue over I-75 at the south end of the wall alignment. The backslope behind the wall will be regraded, where necessary, to a maximum backslope of 2:1 (H:V).

Based upon proposed plan and cross section information provided by the Rii design team, wall heights along the alignment are anticipated to range between 10.2 feet and 13.1 feet. In general, the typical width of a CIP wall foundation (B) is equal to 50 to 70 percent the wall height.

The anticipated bearing materials along the proposed alignment will primarily consist of medium dense to dense gravel and gravel and sand (ODOT A-1-a, A-1-b). CIP wall foundations bearing on these soils or engineered fill, placed and compacted as described in Section 5.3 of the full report, may be proportioned for a nominal bearing resistance as indicated in Table 4. A geotechnical resistance factor of $\phi_b=0.55$ was considered in calculating the factored nominal bearing resistance at the strength limit state.

Retaining Wall K Spread Footing Design Parameters

From Station ¹	To Station ¹	Representative Borings	Maximum Wall Height (ft)	Required Foundation Width ²	Nominal Bearing Resistance (ksf)		Settlement (in)
					Service	Strength ³	
491+73	496+20	B-040 through B-043	13.1	0.70(H)	9.9	29.7	< 0.5

1. Limits of wall determined from plan information provided by the Rii design team. Stationing listed is referenced to the proposed centerline of Ramp C.
2. The required foundation width is expressed as a percentage of the maximum wall height, H.
3. The nominal bearing resistance at the strength limit state is unfactored. Rii recommends that a resistance factor of $\phi_b=0.55$ be considered when calculating the factored nominal bearing resistance at the strength limit state.

Rii performed a verification of the bearing pressure exerted on the subgrade material for the maximum specified wall height. Based on the minimum footing width presented above, the bearing pressure exerted at the front of the wall **will not exceed** the factored nominal bearing resistance at the strength limit state.

For CIP walls bearing on soil, the limiting eccentricity is one-fourth of the base width of the wall. Based on the soil parameters listed in Section 5.1.1 of the full report, for a CIP wall designed with a minimum footing width as noted above, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

Based on the soil parameters listed in Section 5.1.1 for the foundation material, a coefficient of sliding friction of 0.62 was utilized for design along the wall alignment. A geotechnical resistance factor of $\phi_{\tau}=1.0$ was considered when calculating the factored shear resistance between the concrete foundation and bearing soil for sliding. Based on the minimum footing width presented above and utilizing the soil parameters listed in Section 5.1.1 of the full report for the retained material, the resultant horizontal forces on the back of the CIP wall **will exceed** the factored shear resistance at the strength limit state. If a shear key is incorporated into the wall design and the shear key is embedded a minimum of 2.0 feet below the bottom of the footing, then the resultant horizontal forces on the back of the CIP wall **will not exceed** the factored shear resistance at the strength limit state. For the analysis, passive pressure was only considered against the embedded shear key, and passive pressure was not considered for the embedment depth of the footing. A geotechnical resistance factor of $\phi_{ep}=0.50$ was considered when calculating the factored passive resistance on the shear key for sliding.

Per Section 11.6.2.3 of the 2010 AASHTO LRFD BDS, overall (global) stability for CIP walls not supporting structural foundations on spread footings is satisfied when a minimum factor of safety of 1.33 is obtained under loading conditions at the service limit state. Based on the soil parameters listed in Section 5.1.1 of the full report, the resulting factor of safety under drained and undrained conditions along the alignments was greater than 1.33.

Please note that this executive summary does not contain all the information presented in the report. The unabridged subsurface exploration report should be read in its entirety to obtain a more complete understanding of the information presented.

1.0 INTRODUCTION

The overall purpose of this project is to provide detailed subsurface information and recommendations for the design and construction of the HAM-75-7.85 project in Hamilton County, Ohio. This project represents the northern portion of HAM-75-2.30 Mill Creek Expressway improvements. The project will consist of roadway improvements, several new retaining walls and bridge replacements along I-75 from Vine Street to State Route 126. The project site is located in the community limits of St. Bernard, Elmwood Place, Roselawn, and Cincinnati, in Hamilton County, Ohio.

This DRAFT report is a presentation of the structure foundation exploration performed for the design and construction of proposed Retaining Wall K as part of the HAM-75-7.85 project. The proposed wall is located as shown on the vicinity map and boring plan presented in Appendix II. It is understood that this wall will be connected to the rear abutment of the proposed HAM-561-7.01 Seymour Avenue over I-75 bridge structure at the south end of the wall alignment and will extend north along the west side of the proposed Ramp C. According to design details provided by EMH&T the rear abutment for the bridge structure is currently proposed to be a full height cast-in-place (CIP) wall type abutment with associated wingwalls extending to the north and south of the structure. To provide continuity, it is understood that a CIP wall type is being considered as the preferred wall type for the entire alignment of Retaining Wall K. The total wall length is approximately 450 lineal feet.

2.0 GEOLOGY AND OBSERVATIONS OF THE PROJECT

2.1 Site Geology

Both the Illinoian and Wisconsinan glaciers advanced over two-thirds of the State of Ohio, leaving behind glacial features such as moraines, kame deposits, lacustrine deposits and outwash terraces. The glacial and non-glacial regions comprise five physiographic sections grouped by age, depositional process and geomorphic occurrence. Physiographically, the site lies within the Illinoian Till Plain of the Till Plains Section. This area is characterized by rolling ground moraine deposits with many buried valleys alternating between broad floodplains and bedrock gorges. The site area contains silty loam till deposited as ground moraine covered with loess and dissected by the modern day Mill Creek. Ground moraines are deposited during the retreat of a glacier which results in an undifferentiated mixture of clay, silt, sand and gravel. The valley area also contains outwash and alluvium which eroded from hills and valleys with moderate relief. Outwash deposits consist of undifferentiated sand and gravel deposited by meltwater in front of glacial ice and often occurs as valley terraces or low plains. Alluvium and alluvial terrace deposits range in composition from silty clay size particles to cobbles, usually deposited in present and former floodplain areas.



Based on Bedrock Geology and Topography Maps of the area, from the Ohio Department of Natural Resources (ODNR), the underlying bedrock consists of the Ordovician-aged Point Pleasant Formation. The Point Pleasant Formation is comprised of interbedded limestone and shale, averaging 60 percent limestone and 40 percent shale, and ranges from 0 to 80 feet thick. The bedrock surface forms a valley roughly beneath, and following, the alignment of Mill Creek which is aligned northeast-to-southwest. I-75 is aligned roughly parallel to this main bedrock valley from the approximate intersection with State Route 126 to the approximate intersection with Regina Graeter Way, and lies just east of the bottom of the bedrock valley. Along the project alignment, the bedrock surface directly beneath I-75 lies along the slope of the bedrock valley and the bedrock surface ranges between approximate elevations of 385 to 425 feet msl. A smaller bedrock valley branches off to the southeast of the bedrock valley that follows Mill Creek just south of the interchange with State Route 562, and runs roughly parallel with Ross Run and generally beneath the SR 562 alignment. Overall, the bedrock surface along the majority of the project alignment slopes downward to the northwest. According to bedrock topography mapping, the depth to top of bedrock in the vicinity of the project ranges from approximately 120 to 170 feet below the existing ground surface. An illustration of the general geology of Ohio is presented in Appendix I.

2.2 Existing Conditions

The site for the proposed Retaining Wall K is located along the west side of I-75, just north of the HAM-561-7.01 Seymour Avenue over I-75 bridge structure. Overall, the project is located approximately 1.2 miles south of the Lockland split. The proposed wall begins at Ramp C Station 491+73 and continues north to Ramp C Station 496+20 along the west side of I-75. The wall will be located along the west side of the proposed Ramp C from Paddock Road to I-75 southbound and will be constructed in lieu of grading the existing slope to minimize the amount of excavation required to accommodate the proposed alignment. The existing I-75 roadway that runs adjacent to the proposed structure is currently a six-lane, asphalt paved road, and the existing ramp from Paddock Road to I-75 southbound is single-lane, asphalt roadway. The proposed Ramp C will be shift up to 40 feet west of the existing ramp to accommodate the new configuration. The terrain west of I-75 is elevated from the existing roadway and the I-75 mainline is generally level between the Seymour Avenue and Paddock Road bridges.

3.0 EXPLORATION

On October 3 and 4, 2011, a total of four (4) structural borings, designated as B-040-0-11 through B-043-0-11, were drilled to depths ranging from 25.0 to 50.0 feet below the ground surface at the locations illustrated on the boring plan presented in Appendix II.



Table 1. Test Boring Summary

Boring Number	Station	Offset	Latitude	Longitude	Ground Elevation (feet)	Boring Depth
B-040-0-11	492+53.18	119.3' Lt.	39.193444319 °N	84.477287988 °W	545.3	50.0
B-041-0-11	493+96.88	121.2' Lt.	39.193737748 °N	84.476949030 °W	538.7	50.0
B-042-0-11	495+29.72	176.1' Lt.	39.194107936 °N	84.476773485 °W	545.0	25.0
B-043-0-11	496+53.86	201.8' Lt.	39.194406205 °N	84.476542990 °W	547.1	25.0

The boring locations were determined and located in the field by Rii representatives. Geographic latitude and longitude coordinates as well as ground surface elevations at the boring locations are included on the boring logs provided in Appendix IV.

The borings were drilled using an all terrain vehicle (ATV)-mounted rotary drilling machine, utilizing a 4.25-inch inside diameter, hollow-stem auger to advance the holes. Standard penetration testing (SPT) and split spoon sampling were performed in the borings at 2.5-foot increments of depth to 30 feet and at 5.0-foot increments thereafter to the boring termination depth. The SPT, per the American Society for Testing and Materials (ASTM) designation D1586, is conducted by letting a 140-pound hammer falling 30.0 inches to drive a 2.0-inch outer diameter split spoon sampler 18.0 inches. Rii utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. Driving resistance is recorded on the boring logs in terms of blows per 6-inch interval of the driving distance. The second and third intervals are added to obtain the number of blows per foot (N). Standard penetration blow counts aid in determining soil properties applicable in foundation system design. Measured blow count (N) values are corrected to an equivalent (60%) energy ratio, N_{60} , by the following equation. Both values are represented on boring logs in Appendix IV.

$$N_{60} = N_m \cdot (ER/60)$$

Where:

N_m = measured N value

ER = drill rod energy ratio, expressed as a percent, for the system used

The hammer for the ATV-mounted drill rig used for this project was calibrated on May 6, 2011, and has a drill rod energy ratio of 77.1 percent.

During drilling, Rii personnel prepared field logs showing the encountered subsurface conditions. Soil samples obtained from the drilling operation were preserved and sealed in glass jars and delivered to the soil laboratory. In the laboratory, the soil samples were visually classified and select samples were tested, as noted in Table 2.



Table 2. Laboratory Test Schedule

Laboratory Test	Test Designation	Number of Tests Performed
Natural Moisture Content	ASTM D 2216	52
Plastic and Liquid Limits	AASHTO T89, T90	11
Sieve/Hydrometers	AASHTO T88	11

The tests performed are necessary to classify existing soil according to the Ohio Department of Transportation (ODOT) classification system and to estimate engineering properties of importance in determining foundation design and construction recommendations. Results of the laboratory testing are presented, in part, on the boring logs in Appendix IV and also in Appendix V. A description of the soil terms used throughout this report is presented in Appendix III.

Hand penetrometer readings, which provide a rough estimate of the unconfined compressive strength of the soil, were reported on the boring logs in units of tons per square foot (tsf) and were utilized to classify the consistency of the cohesive soil in each layer. An indirect estimate of the unconfined compressive strength of the cohesive split spoon samples can also be made from a correlation with the blow counts (N_{60}). Please note that split spoon samples are considered to be disturbed and the laboratory determination of their shear strengths may vary from undisturbed conditions.

4.0 FINDINGS

Interpreted engineering logs have been prepared from the field logs, visual examination of samples, and laboratory testing. Classification follows the current version of the ODOT Specification for Geotechnical Exploration (SGE). The following is a summary of what was found in the test borings and what is represented on the boring logs.

4.1 Surficial Material

All of the borings were drilled along the existing embankment adjacent to the I-75 southbound lanes and encountered 4.0 to 8.0 inches of topsoil at the existing ground surface, identified by the significant presence of organic matter and vegetation.

4.2 Subsurface Soils

Beneath the surface material in borings B-040 and B-043, material identified as existing fill was encountered extending to a depth of 8.0 and 5.5 feet below the ground surface, respectively. The fill material consisted of dark brown and brown gravel and sand (ODOT A-1-b) and contained brick fragments.



Underlying the surface materials and fill material in borings B-040 and B-043, natural soils were encountered consisting of granular soils overlying cohesive soils. The granular soils were generally described as brown and brownish gray gravel, gravel and sand, gravel with sand and silt and fine sand (ODOT A-1-a, A-1-b, A-2-4, A-3). The cohesive soils were generally described as gray, brownish gray and brown silt and clay and silty clay (ODOT A-6a, A-6b) with lesser percentages of gravel and/or sand.

The relative density of granular soils is primarily derived from SPT blow counts (N_{60}). Based on the SPT blow counts obtained, the granular soil encountered ranged from loose ($5 \leq N_{60} \leq 10$ blows per foot [bpf]) to very dense ($N_{60} > 50$ bpf). Overall blow counts recorded from the SPT sampling ranged from 9 to 69 bpf. The shear strength and consistency of the cohesive soils are primarily derived from the hand penetrometer values (HP). The cohesive soil encountered ranged from stiff ($1.0 < \text{HP} \leq 2.0$ tsf) to very stiff ($2.0 < \text{HP} \leq 4.0$ tsf). The unconfined compressive strength of the cohesive soil samples tested, obtained from the hand penetrometer, ranged from 1.5 to 3.75 tsf.

Natural moisture contents of the soil samples tested ranged from 2 to 28 percent. The natural moisture content of the cohesive soil samples tested for plasticity index ranged from 2 percent below to 5 percent above their corresponding plastic limits. The moisture contents of the native soils are generally considered to be slightly below to moderately above the optimum moisture levels.

4.3 Bedrock

Bedrock was not encountered in any of the borings performed for this exploration.

4.4 Groundwater

Groundwater was encountered initially during the drilling process in borings B-040 and B-041 at a depth of 36.0 and 27.0 feet below the ground surface, respectively. The groundwater level at the completion of drilling could not be obtained due to sealing the boreholes with grout. Please note that short-term water level readings, especially in cohesive soils, are not necessarily an accurate indication of the actual groundwater level. In addition, groundwater levels or the presence of groundwater are considered to be dependent on seasonal fluctuations in precipitation.

A more comprehensive description of what was encountered during the drilling process may be found on the boring logs in Appendix IV.

5.0 ANALYSES AND RECOMMENDATIONS

Data obtained from the drilling and testing program have been used to determine the foundation support capabilities and the settlement potential for the soil encountered at the site. These parameters have been used to provide guidelines for the design of foundation systems for the subject retaining wall, as well as the construction specifications related to the placement of foundation systems and general earthwork recommendations, which are discussed in the following paragraphs.

Design details of the proposed retaining wall were provided by the Rii design team. It is understood that the proposed retaining wall will be a reinforced concrete cast-in-place (CIP) wall type that will connect to the northwest wingwall of the rear abutment of the proposed HAM-561-0701 bridge carrying Seymour Avenue over I-75 at the south end of the wall alignment. The backslope behind the wall will be regraded, where necessary, to a maximum backslope of 2:1 (H:V).

5.1 Retaining Wall Recommendations

Based upon proposed plan and cross section information provided by the Rii design team, wall heights along the alignment are anticipated to range between 10.2 feet and 13.1 feet. For CIP walls bearing on earthen foundations, footings should be proportioned such that the equivalent bearing pressure exerted at the front of the wall will not exceed the factored nominal bearing resistance at the strength limit state. Further, the footings should also be proportioned such that the entire footing width remains in compression (no tensile stresses form under the footing, pulling the footing up and away from the bearing surface). It is understood that the foundations for CIP walls will bear approximately 3.5 feet below the finished grade.

In general, the typical width of a CIP wall foundation (B) is equal to 50 to 70 percent the wall height. For the analysis, the foundation width was varied, starting at 50 percent of the wall height, until external and global stability requirements were satisfied for various wall heights and subsurface conditions.

It is recommended that the foundation surface for the CIP wall be observed by a geotechnical engineer or representative thereof to verify that suitable bearing material exists across the entire surface. Unstable soils, primarily those containing silt (ODOT A-4b), if encountered, may be stabilized as noted below:

- Undercut 24 inches +/- pending results of proof-roll;
- Place ODOT Item 712.09 Type D Geotextile Fabric;
- Placement of 24 inches of 703.16C Type C Granular Fill.



5.1.1 Strength Parameters Utilized in External and Global Stability Analyses

The shear strength parameters utilized in the external and global stability analysis of the retaining wall are provided in Table 3. **Error! Reference source not found..**

Table 3. Shear Strength Parameters Utilized in Stability Analyses

Material Type	Unit Weight, γ (pcf)	Effective Friction Angle, ϕ' (°)	Effective Cohesion, c' (psf)	Undrained Shear Strength, S_u (psf)
Medium Dense Gravel and Gravel and Sand (ODOT A-1-a, A-1-b)	125 to 130	30 to 32	0	N/A
Very Stiff Clay (ODOT A-7-6) ¹	120	21	0	2,500

1. Based on laboratory consolidated undrained triaxial testing performed on an undisturbed samples from the HAM-75-12.60 Part I project

The long term shear strength parameters (drained/effective stress) for the natural cohesive soils were determined from consolidated undrained (CU) triaxial compression tests performed on undisturbed samples from the HAM-75-12.60 project that is adjacent to this project location to the north. The undrained shear strength for the clay is based on hand penetrometer values performed on the recovered samples. The friction angle for the natural granular soils encountered was determined based on correlations with the N_{60} value from the SPT testing of the soil. The laboratory test results referenced above are provided in Appendix V.

5.1.2 Bearing Stability

The anticipated bearing materials along the proposed alignment will primarily consist of medium dense to dense gravel and gravel and sand (ODOT A-1-a, A-1-b). CIP wall foundations bearing on these soils or engineered fill, placed and compacted as described in Section 5.3, may be proportioned for a nominal bearing resistance as indicated in Table 4. A geotechnical resistance factor of $\phi_b=0.55$ was considered in calculating the factored nominal bearing resistance at the strength limit state. The foundation widths presented in the following tables are the minimum footing widths required to satisfy external and global stability requirements. Based on cross section information provided by the Rii design team, there will be little to no change between the proposed profile grade and existing profile grade behind the wall. Therefore, little to no settlement is anticipated under the loading from the proposed wall along the alignment.

Table 4. Retaining Wall K Spread Footing Design Parameters

From Station ¹	To Station ¹	Representative Borings	Maximum Wall Height (ft)	Required Foundation Width ²	Nominal Bearing Resistance (ksf)		Settlement (in)
					Service	Strength ³	
491+73	496+20	B-040 through B-043	13.1	0.70(H)	9.9	29.7	< 0.5

1. Limits of wall determined from plan information provided by the Rii design team. Stationing listed is referenced to the proposed centerline of Ramp C.
2. The required foundation width is expressed as a percentage of the maximum wall height, H.
3. The nominal bearing resistance at the strength limit state is unfactored. Rii recommends that a resistance factor of $\phi_b=0.55$ be considered when calculating the factored nominal bearing resistance at the strength limit state.

Rii performed a verification of the bearing pressure exerted on the subgrade material for the maximum specified wall height. Based on the minimum footing width presented, the bearing pressure exerted at the front of the wall **will not exceed** the factored nominal bearing resistance at the strength limit state.

5.1.3 Sliding Stability

The resistance of the CIP wall to sliding will be dependent on the friction between the concrete and the bearing soil per Section 11.6.3.6 of the 2010 AASHTO LRFD BDS. For CIP walls, sliding resistance is determined by multiplying a coefficient of sliding friction “f” times the total vertical force at the base of the wall. The coefficient of sliding friction is determined based on the friction angle of the foundation soil. Based on the soil parameters listed in Section 5.1.1 for the foundation material, a coefficient of sliding friction of 0.62 was utilized for design. A geotechnical resistance factor of $\phi_\tau=1.0$ was considered when calculating the factored shear resistance between the concrete foundation and bearing soil for sliding. Based on the minimum footing width presented in Table 4 and utilizing the soil parameters listed in Section 5.1.1 for the retained material, the resultant horizontal forces on the back of the CIP wall **will exceed** the factored shear resistance at the strength limit state.

If a shear key is incorporated into the wall design and the shear key is embedded a minimum of 2.0 feet below the bottom of the footing, then the resultant horizontal forces on the back of the CIP wall **will not exceed** the factored shear resistance at the strength limit state. For the analysis, passive pressure was only considered against the embedded shear key, and passive pressure was not considered for the embedment depth of the footing. A geotechnical resistance factor of $\phi_{ep}=0.50$ was considered when calculating the factored passive resistance on the shear key for sliding.

5.1.4 Eccentricity (Overturning Stability)

The resistance of the CIP wall to overturning will be dependent on the on the location of the resultant force at the bottom of the wall due to the overturning and resisting moments acting on the wall. For CIP walls, overturning stability is determined by calculating the eccentricity of the resultant force from the midpoint of the base of the wall and comparing this value to a limiting eccentricity value. Per Section 11.10.5.5 of the 2010 AASHTO LRFD BDS, for CIP wall foundations bearing on soil, the location of the resultant of the reaction forces shall be within the middle one-half of the base width. Therefore, the limiting eccentricity is one-fourth of the base width of the wall. Rii performed a verification of the eccentricity of the resultant force for the maximum specified wall height indicated in Table 4. Based on the required foundation width presented in Table 4 and utilizing the soil parameters listed in Section 5.1.1 for the retained material, the calculated eccentricity of the resultant force **will not exceed** the limiting eccentricity at the strength limit state.

5.1.5 Global (Overall) Stability

A slope stability analysis was performed to check the global stability of the wall along the alignment. As per AASHTO LRFD BDS, safety against soil failure shall be evaluated at the service limit state by assuming the concrete and soil backfill to be a rigid body. Soil parameters utilized in external stability analyses are presented Section 5.1.1. For the global stability condition, it was considered that the failure plane will not cross through any portion of the supported soil mass above the concrete or through the concrete footing itself.

Per Section 11.6.2.3 of the 2010 AASHTO LRFD BDS, overall (global) stability for CIP walls not supporting structural foundations on spread footings is satisfied if the product of the factor of safety from the slope stability output multiplied by the resistance factor $\phi=0.75$ is greater than 1.0. Therefore, global stability is satisfied when a minimum factor of safety of 1.33 is obtained. Based on the recommended footing dimensions listed in Table 4, the resulting factor of safety under drained conditions (long-term stability) along the alignment was greater than 1.33. The wall was also evaluated under undrained conditions (short-term stability) to verify the stability of the wall during and immediately following construction. The resulting factor of safety along the alignment under undrained conditions was also greater than 1.33.

Calculations for external (bearing and sliding resistance and limiting eccentricity) and overall (global) stability of the CIP walls are provided in Appendix VI.



5.2 Lateral Earth Pressure

For the soil types encountered in the borings, the “in-situ” unit weight (γ), cohesion (c), effective angle of friction (ϕ'), and lateral earth pressure coefficients for at-rest conditions (k_o), active conditions (k_a), and passive conditions (k_p) have been estimated and are provided in Table 5 and Table 6.

Table 5. Estimated Undrained (Short-term) Soil Parameters for Design

Soil Type	γ (pcf) ¹	c (psf)	ϕ	k_a	k_o	k_p
Stiff Cohesive Soil	115	1,500	0°	1.0	1.0	1.0
Very Stiff Cohesive Soil	120	2,500	0°	1.0	1.0	1.0
Loose Granular Soil	120	0	29°	0.34	0.52	2.88
Medium Dense to Dense Granular Soil	125	0	32°	0.31	0.47	3.25
Very Dense Granular Soil	135	0	35°	0.27	0.42	3.69
Compacted Cohesive Engineered Fill	125	1,500	0°	1.0	1.0	1.0
Compacted Granular Engineered Fill	135	0	33°	0.30	0.46	3.39

1. When below groundwater table, use effective unit weight, $\gamma' = \gamma - 62.4$ pcf and add hydrostatic water pressure.

Table 6. Estimated Drained (Long-term) Soil Parameters for Design

Soil Type	γ (pcf) ¹	c (psf)	ϕ	k_a	k_o	k_p
Natural Cohesive Soil	115	0	27°	0.38	0.54	2.66
Loose Granular Soil	120	0	29°	0.34	0.52	2.88
Medium Dense to Dense Granular Soil	125	0	32°	0.31	0.47	3.25
Very Dense Granular Soil	135	0	35°	0.27	0.42	3.69
Compacted Cohesive Engineered Fill	125	0	28°	0.36	0.53	2.77
Compacted Granular Engineered Fill	135	0	33°	0.30	0.46	3.39

1. When below groundwater table, use effective unit weight, $\gamma' = \gamma - 62.4$ pcf and add hydrostatic water pressure.

These parameters are considered appropriate for the design of subsurface walls and excavation support systems. It is recommended that the retaining wall structure be designed based on active conditions. The values in this table have been estimated from correlation charts based on minimum standards specified for compacted engineered fill materials. These recommendations do not take into consideration the effect of any surcharge loading or a sloped ground surface (a flat surface is considered based on cross section information provided by the Rii design team). Earth pressures on

excavation support systems will be dependent on the type of sheeting and method of bracing or anchorage.

5.3 Construction Considerations

All site work shall conform to local codes and to the latest ODOT Construction and Material Specifications (CMS), including that all excavation and embankment preparation and construction should follow ODOT Item 200 (Earthwork).

Fill soil placed for foundation support should be placed in loose lifts not to exceed 8.0 inches. Fill soil placed under structures shall be compacted to not less than 100 percent of the maximum dry density obtained by the Standard Proctor Test (ASTM D698). Fill soil containing excess moisture shall be required to dry prior to or during compaction to a moisture content not greater than 3.0 percent above or below optimum. However, for material that displays pronounced elasticity or deformation under the action of loaded rubber tire construction equipment, the moisture content shall be reduced to optimum if necessary to secure stability. Drying of wet soil shall be expedited by the use of plows, discs, or by other approved methods when so ordered by the site geotechnical engineer.

Generally, materials utilized for engineered fill should free of waste construction debris and other deleterious materials and meet the following requirements:

- Maximum Dry Density per ASTM D698 > 110 pcf
- Liquid Limit < 40
- Plasticity Index < 15
- Organic Matter < 3 percent
- Maximum Particle Size < 3 inches
- Silt Content (between 0.075 and 0.005 mm) < 45 percent

Compacted granular fill shall meet the above specification and additionally shall have a maximum 35 percent passing the No. 200 sieve.

5.3.1 Excavation Considerations

All excavations should be shored / braced or laid back at a safe angle in accordance to Occupational Safety and Health Administration (OSHA) guidelines. During excavation, if slopes cannot be laid back to OSHA Standards due to adjacent structures or other obstructions, sheeting boxes may be required. The following table should be utilized as a general guide for implementing OSHA guidelines when estimating excavation back slopes at the various boring locations. Actual excavation back slopes must be field verified by qualified personnel at the time of excavation in strict accordance with OSHA guidelines.



Table 7. Excavation Back Slopes

Soil	Maximum Back Slope	Notes
Soft to Medium Stiff Cohesive	1.5 : 1.0	Above Ground Water Table and No Seepage
Stiff Cohesive	1.0 : 1.0	Above Ground Water Table and No Seepage
Very Stiff to Hard Cohesive	0.75 : 1.0	Above Ground Water Table and No Seepage
All Granular & Cohesive Soil Below Ground Water Table or with Seepage	1.5 : 1.0	None
Rock to 3.0' +/- below Auger Refusal	0.75 : 1.0	Above Ground Water Table and No Seepage
Stable Rock	Vertical	Above Ground Water Table and No Seepage

5.3.2 Groundwater Considerations

Based on the groundwater observations made during drilling, groundwater seepage may be encountered during installation of the drilled shafts. Where/if groundwater is encountered, proper groundwater control should be employed and maintained to prevent disturbance to excavation bottoms consisting of cohesive soil, and to prevent the possible development of a quick or "boiling" condition where soft silts and/or fine sands are encountered. It is preferable that the groundwater level, if encountered, be maintained at least 36 inches below the deepest excavation. Any seepage or groundwater encountered at this site should be able to be controlled by pumping from temporary sumps. Additional measures may be required depending on seasonal fluctuations of the groundwater level. Note that determining and maintaining actual groundwater levels during construction is the responsibility of the contractor.

6.0 LIMITATIONS OF STUDY

The above recommendations are predicated upon construction inspection by a qualified soil technician under the direct supervision of a professional geotechnical engineer. Adequate testing and inspection during construction are considered necessary to assure an adequate foundation system and are part of our recommendations.

Our recommendations for this project were developed utilizing soil and bedrock information obtained from the test borings that were made at the proposed site. At this time we would like to point out that soil borings only depict the soil and bedrock conditions at the specific locations and time at which they were made. The conditions at other locations on the site may differ from those occurring at the boring locations.

The conclusions and recommendations herein have been based upon the available soil and bedrock information and the design details furnished by a representative of the owner of the proposed project. Any revision in the plans for the proposed construction from those anticipated in this report should be brought to the attention of the geotechnical engineer to determine whether any changes in the foundation or earthwork recommendations are necessary. If deviations from the noted subsurface conditions are encountered during construction, they should also be brought to the attention of the geotechnical engineer.

The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the site studied. Any statements in this report or on the test boring logs regarding odors, staining of soils, or other unusual conditions observed are strictly for the information of our client.

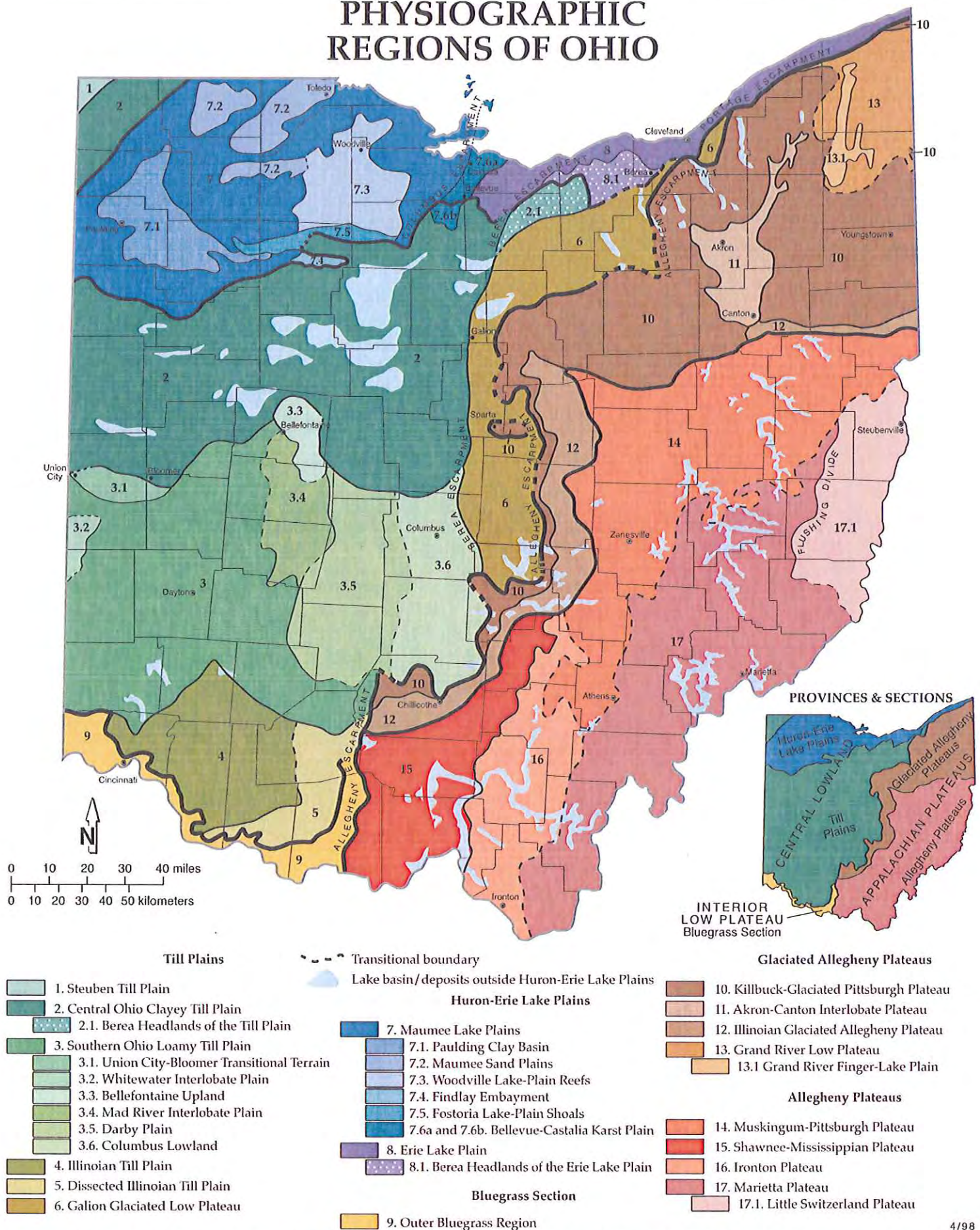
Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. Resource International is not responsible for the conclusions, opinions, or recommendations made by others based upon the data included



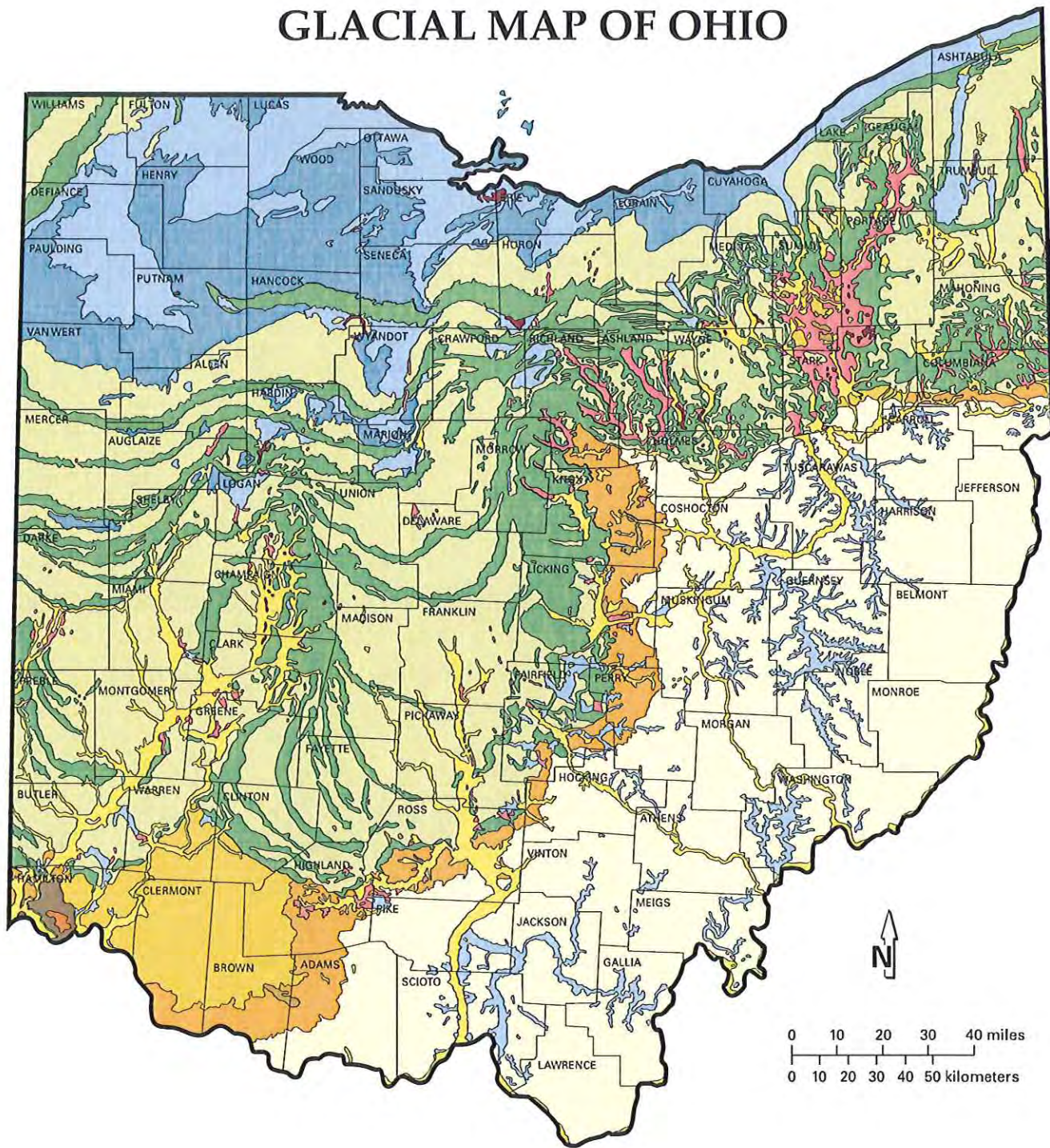
APPENDIX I

STATE GEOLOGY

PHYSIOGRAPHIC REGIONS OF OHIO



GLACIAL MAP OF OHIO



0 10 20 30 40 miles
0 10 20 30 40 50 kilometers

WISCONSINAN
(14,000 to 24,000 years old)

- Ground moraine
- Wave-planed ground moraine
- End moraine

ILLINOIAN
(130,000 to 300,000 years old)

- Ground moraine
- Dissected ground moraine
- Hummocky moraine

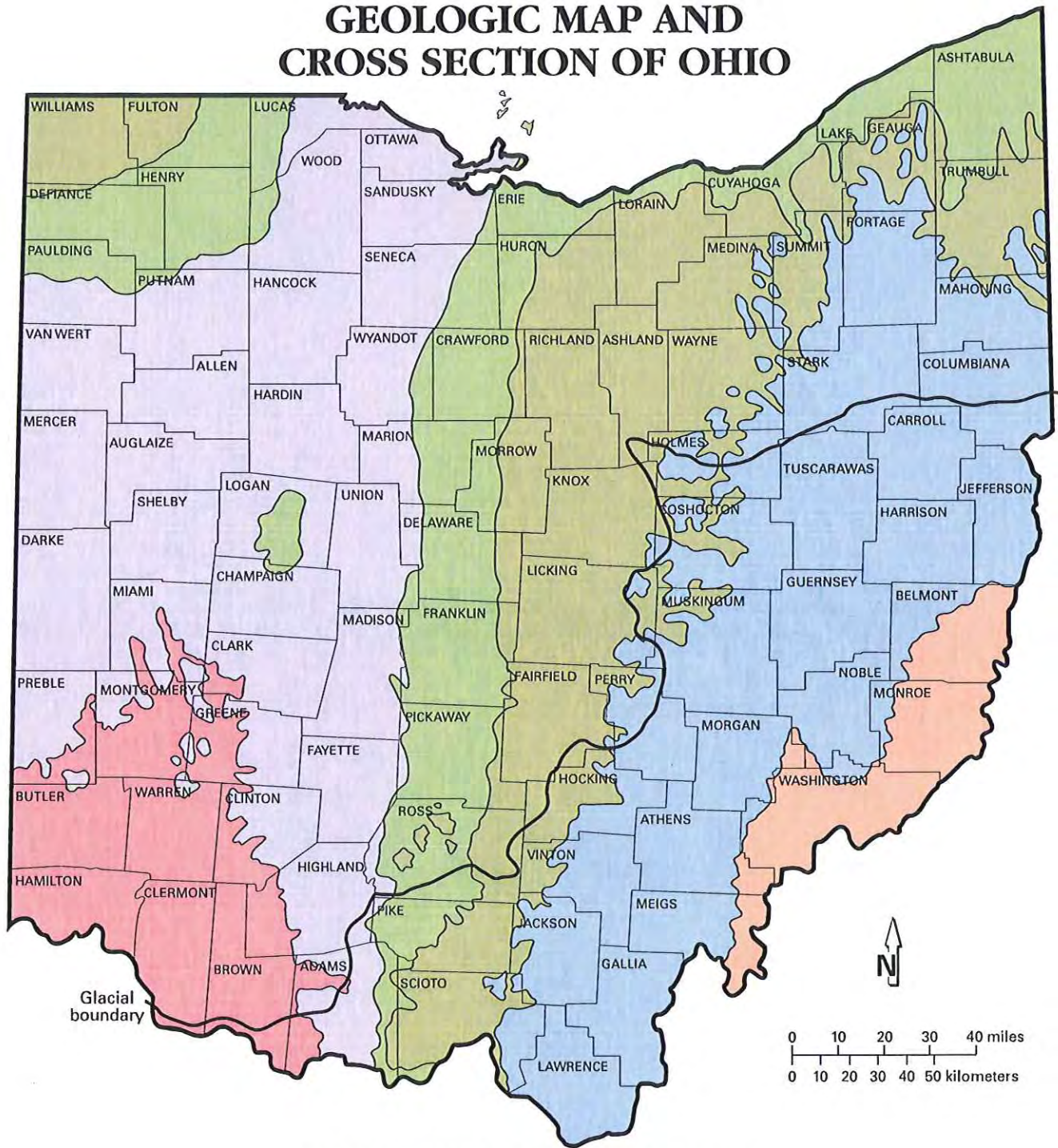
PRE-ILLINOIAN
(older than 300,000 years)

- Ground moraine
- Dissected ground moraine

- Kames and eskers
- Outwash
- Lake deposits
- Peat
- Colluvium

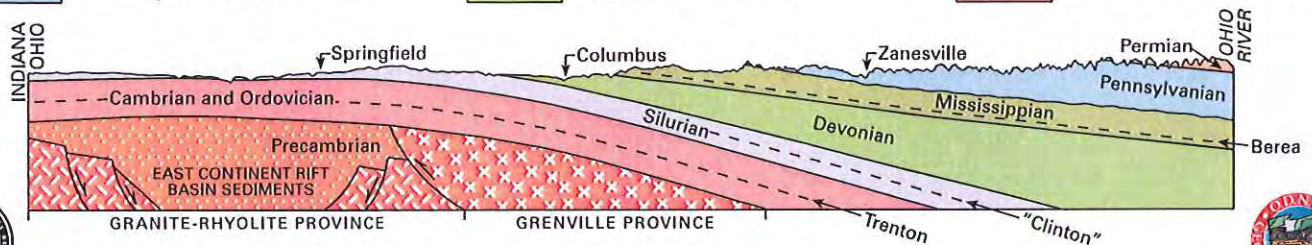


GEOLOGIC MAP AND CROSS SECTION OF OHIO



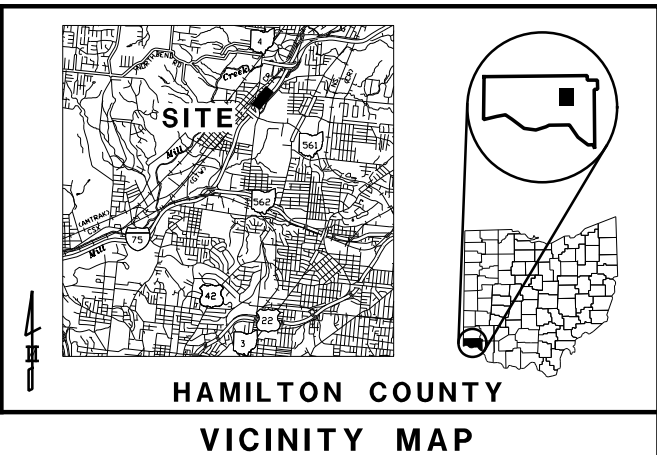
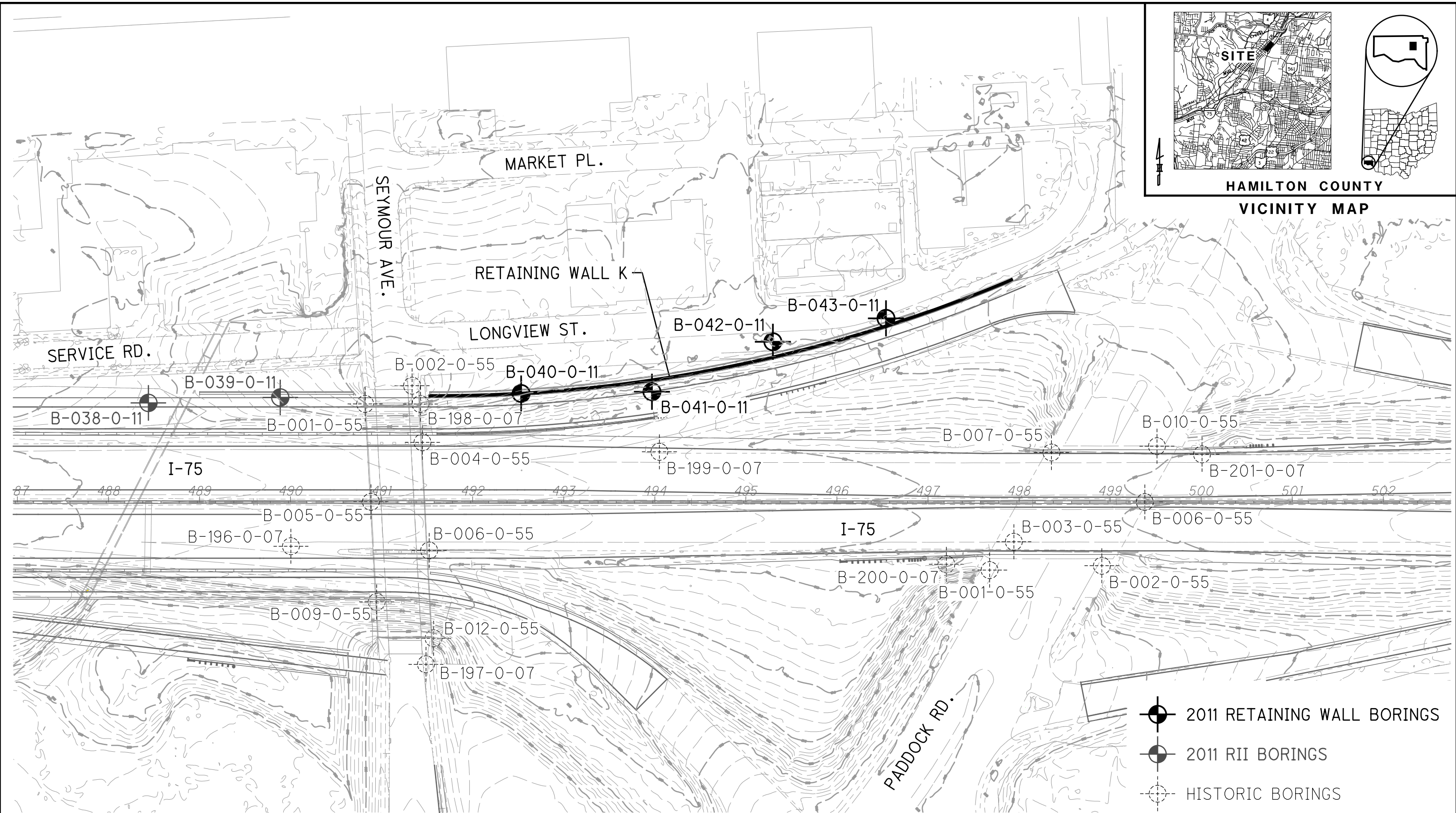
GEOLOGIC SYSTEM (million years before present)

Permian (286-245)	Mississippian (360-320)	Silurian (438-408)
Pennsylvanian (320-286)	Devonian (408-360)	Ordovician (505-438)



APPENDIX II

VICINITY MAP AND BORING PLAN



BORING PLAN
HAM-75-7.85 RETAINING WALL K
HAMILTON COUNTY, OHIO

PROJECT NO. Rii B-10-020	DRAWN RRM	REVIEWED BRT
SCALE: 1"=100'		DATE 6-13-12


RESOURCE
INTERNATIONAL, INC.

APPENDIX III

DESCRIPTION OF SOIL TERMS

DESCRIPTION OF SOIL TERMS

The following terminology was used to describe soils throughout this report and is generally adapted from ASTM 2487/2488 and ODOT Specifications for Geotechnical Explorations.

Granular Soils - The relative compactness of granular soils is described as:

ODOT A-1, A-2, A-3, A-4 (non-plastic) or USCS GW, GP, GM, GC, SW, SP, SM, SC, ML (non-plastic)

<u>Description</u>	<u>Blows per foot – SPT (N₆₀)</u>		
Very Loose	Below		5
Loose	5	-	10
Medium Dense	11	-	30
Dense	31	-	50
Very Dense	Over		50

Cohesive Soils - The relative consistency of cohesive soils is described as:

ODOT A-4, A-5, A-6, A-7, A-8 or USCS ML, CL, OL, MH, CH, OH, PT

<u>Description</u>	<u>Blows per foot – SPT (N₆₀)</u>			<u>Unconfined Compression (tsf)</u>
Very Soft	Below		2	UCS ≤ 0.25
Soft	2	-	4	0.25 < UCS ≤ 0.5
Medium Stiff	5	-	8	0.5 < UCS ≤ 1.0
Stiff	9	-	15	1.0 < UCS ≤ 2.0
Very Stiff	16	-	30	2.0 < UCS ≤ 4.0
Hard	Over		30	UCS > 4.0

Gradation - The following size-related denominations are used to describe soils:

<u>Soil Fraction</u>	<u>USCS Size</u>	<u>ODOT Size</u>
Boulders	Larger than 12"	Larger than 12"
Cobbles	12" to 3"	12" to 3"
Gravel coarse	3" to ¾"	3" to ¾"
fine	¾" to 4.75 mm (¾" to #4 Sieve)	¾" to 2.0 mm (¾" to #10 Sieve)
Sand coarse	4.75 mm to 2.0 mm (#4 to #10 Sieve)	2.0 mm to 0.42 mm (#10 to #40 Sieve)
medium	2.0 mm to 0.42 mm (#10 to #40 Sieve)	-
fine	0.42 mm to 0.074 mm (#40 to #200 Sieve)	0.42 mm to 0.074 mm (#40 to #200 Sieve)
Silt	0.074 mm to 0.005 mm (#200 to 0.005 mm)	0.074 mm to 0.005 mm (#200 to 0.005 mm)
Clay	Smaller than 0.005 mm	Smaller than 0.005 mm

Modifiers of Components - Modifiers of components are as follows:

<u>Term</u>	<u>Range</u>		
Trace	0%	-	10%
Little	10%	-	20%
Some	20%	-	35%
And	35%	-	50%

Moisture Table - The following moisture-related denominations are used to describe cohesive soils:

<u>Term</u>	<u>Range - USCS</u>	<u>Range - ODOT</u>
Dry	0% to 10%	Well below Plastic Limit
Damp	>2% below Plastic Limit	Below Plastic Limit
Moist	2% below to 2% above Plastic Limit	Above PL to 3% below LL
Very Moist	>2% above Plastic Limit	
Wet	≥ Liquid Limit	3% below LL to above LL

Organic Content – The following terms are used to describe organic soils:

<u>Term</u>	<u>Organic Content (%)</u>
Slightly organic	2-4
Moderately organic	4-10
Highly organic	>10

Bedrock – The following terms are used to describe bedrock hardness:





















<u>Term</u>	<u>Blows per foot – SPT (N)</u>		
Very Soft	Below		50
Soft	50/5"	–	50/6"
Medium Hard	50/3"	–	50/4"
Hard	50/1"	–	50/2"
Very Hard	50/0"		



CLASSIFICATION OF SOILS

Ohio Department of Transportation

(The classification of a soil is found by proceeding from top to bottom of the chart.
The first classification that the test data fits is the correct classification.)

SYMBOL	DESCRIPTION	Classification		LL _O /LL x 100*	% Pass #40	% Pass #200	Liquid Limit (LL)	Plastic Index (PI)	Group Index Max.	REMARKS
		AASHTO	OHIO							
	Gravel and/or Stone Fragments	A-1-a			30 Max.	15 Max.		6 Max.	0	Min. of 50% combined gravel, cobble and boulder sizes
	Gravel and/or Stone Fragments with Sand	A-1-b			50 Max.	25 Max.		6 Max.	0	
	Fine Sand	A-3			51 Min.	10 Max.	NON-PLASTIC		0	
	Coarse and Fine Sand	--	A-3a			35 Max.		6 Max.	0	Min. of 50% combined coarse and fine sand sizes
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4				35 Max.	40 Max.	10 Max.	0	
		A-2-5					41 Min.			
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6				35 Max.	40 Max.	11 Min.	4	
		A-2-7					41 Min.			
	Sandy Silt	A-4	A-4a	76 Min.		36 Min.	40 Max.	10 Max.	8	Less than 50% silt sizes
	Silt	A-4	A-4b	76 Min.		50 Min.	40 Max.	10 Max.	8	50% or more silt sizes
	Elastic Silt and Clay	A-5		76 Min.		36 Min.	41 Min.	10 Max.	12	
	Silt and Clay	A-6	A-6a	76 Min.		36 Min.	40 Max.	11 - 15	10	
	Silty Clay	A-6	A-6b	76 Min.		36 Min.	40 Max.	16 Min.	16	
	Elastic Clay	A-7-5		76 Min.		36 Min.	41 Min.	≤ LL-30	20	
	Clay	A-7-6		76 Min.		36 Min.	41 Min.	> LL-30	20	
	Organic Silt	A-8	A-8a	75 Max.		36 Min.				W/o organics would classify as A-4a or A-4b
	Organic Clay	A-8	A-8b	75 Max.		36 Min.				W/o organics would classify as A-5, A-6a, A-6b, A-7-5 or A-7-6
MATERIAL CLASSIFIED BY VISUAL INSPECTION										
	Sod and Topsoil			Uncontrolled Fill (Describe)			Bouldery Zone			Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc
	Pavement or Base									

* Only perform the oven-dried liquid limit test and this calculation if organic material is present in the sample.

APPENDIX IV

BORING LOGS:

B-040-0-11 through B-043-0-11

Definitions of Abbreviations for Boring Logs


A	=	Adhesion (pounds per square foot)
AS	=	Auger Sample
BCP	=	Bentonite Chips or Pellets
C	=	Cohesion (pounds per square foot)
CB	=	Cased (Concentric) Boring
C/B	=	Neat Cement/Bentonite Grout
Cl ⁻	=	Chloride Ion Concentration (parts per million)
FA	=	Angle of Internal Friction (degrees)
FF	=	Friction Factor
GS	=	Geoprobe Sample
HSA	=	Hollow Stem Auger
HSB	=	High Solids Content Bentonite Grout
K	=	Modulus of Horizontal Subgrade Reaction (kips per cubic foot)
LOI	=	Percent Organic Content (by weight) as determined by ASTM D-2974 (loss on ignition test)
MD	=	Rotary Mud Drilling
NQ	=	Wireline Method (1.875-inch diameter rock core)
NX	=	Conventional Method (2.126-inch diameter rock core)
PC	=	Neat Portland Cement Grout
PID	=	Photo-Ionization Detector Reading (parts per million)
qh	=	Unconfined Compressive Strength of Soil as determined by a hand penetrometer (tons per square foot)
qr	=	Unconfined Compressive Strength of Intact Rock Core as determined by ASTM D-2938 (pounds per square inch)
qu	=	Unconfined Compressive Strength of Soil as determined by ASTM D-2166 (tons per square foot)
quu	=	Unconsolidated-Undrained Triaxial Compressive Strength as determined by ASTM D-2850 (pounds per square foot)
RC	=	Rock Coring
SO ⁴⁻	=	Sulfate Concentration
SFA	=	Solid Flight Auger
SS	=	Split Spoon Sample
3S	=	For instances of no recovery from standard SS interval, a 3.0 inch O.D. split spoon is driven the full length of the standard SS interval plus an additional 6.0 inches to obtain a representative sample. Only the final 6.0 inches of sample is retained. Blow counts from 3S sampling are not correlated with N ₆₀ values.
ss	=	Soluble Salts (conductivity)
ST	=	Thin-walled (Shelby) Tube Sample
uw	=	"In-Situ" Unit Weight of Soil (pounds per cubic foot)
VIS	=	Visual classification only, no testing performed
WOH	=	Weight of Hammer and Drill Rods "pushed" split spoon sampler 6-inches.
WD	=	Rotary Wash Drilling

RESOURCE INTERNATIONAL, INC.[illegible]

[illegible]

2010 ODOT BORING LOG-RIL-WITH LAT/LONG - OH DOT.GDT - 06/15/12 18:01 - \C\EDC\RILEVELAND.RESOURCE\INTERNATIONAL\COM\FOUR\GINT8\PROJECTS\2010\B-10-020\B-10-020 B-040


RESOURCE INTERNATIONAL, INC.

	PROJECT: HAM-75-7.85	DRILLING FIRM / OPERATOR: RII / T.F.	DRILL RIG: CME-750X (SN 310218)	STATION / OFFSET: 495+29.72 / 176.1' Lt	EXPLORATION ID B-042-0-11
	TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: RII / S.M.	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED CL I-75	
	PID: 77889 BR ID: NA	DRILLING METHOD: 4.25" HSA	CALIBRATION DATE: 5/6/11	ELEVATION: 545.0 (MSL) EOB: 25.0 ft.	PAGE 1 OF 1
	START: 10/4/11 END: 10/4/11	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.1	LAT / LONG : 39.194107936 ° N / 84.476773485 ° W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
0.3' - TOPSOIL (4.0")	545.0																<V>	
MEDIUM DENSE, BROWNISH GRAY GRAVEL AND SAND , TRACE SILT, TRACE CLAY, DAMP.	544.7	1	5														<V>	
		2	4	12	50	SS-1	-	-	-	-	-	-	-	-	4	A-1-b (V)	<V>	
	542.0	3															<V>	
MEDIUM DENSE TO VERY DENSE, BROWNISH GRAY GRAVEL AND SAND , TRACE SILT, TRACE CLAY, DRY TO DAMP.		4	4	31	44	SS-2	-	-	-	-	-	-	-	-	4	A-1-b (V)	<V>	
		5															<V>	
		6	14														<V>	
		7	21	53	78	SS-3	-	38	31	22	8	1	NP	NP	NP	5	A-1-b (0)	
		8															<V>	
		9	5														<V>	
		10	12	30	78	SS-4	-	-	-	-	-	-	-	-	6	A-1-b (V)	<V>	
-COBBLES PRESENT THROUGHOUT		11															<V>	
		12	8														<V>	
		13	10	31	67	SS-5	-	-	-	-	-	-	-	-	5	A-1-b (V)	<V>	
		14															<V>	
		15	14														<V>	
		16	17	46	33	SS-6	-	-	-	-	-	-	-	-	3	A-1-b (V)	<V>	
		17															<V>	
		18	14														<V>	
		19	7	24	78	SS-7	-	-	-	-	-	-	-	-	5	A-1-b (V)	<V>	
		20															<V>	
		21															<V>	
		22	9														<V>	
		23	16	37	78	SS-8	-	32	32	25	9	2	NP	NP	NP	5	A-1-b (0)	
		24															<V>	
		25	12														<V>	
		26	29	69	83	SS-9	-	-	-	-	-	-	-	-	3	A-1-b (V)	<V>	
	522.0	27															<V>	
MEDIUM DENSE, BROWNISH GRAY GRAVEL WITH SAND, SILT, AND CLAY , MOIST.		28	5														<V>	
		29	7	24	72	SS-10	-	-	-	-	-	-	-	-	12	A-2-6 (V)	<V>	
	520.0	30															<V>	
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		120															<V>	

2010 ODOT BORING LOG-RIL-WITH LAT/LONG - OH DOT GDT - 06/15/12 18:08 - \C\EDC\RILEVELAND.RESOURCEINTERNATIONAL.COM\FOUR\GINT8\PROJECTS\2010\B-10-020\B-10-020 B-040

RESOURCE INTERNATIONAL, INC.

	PROJECT: HAM-75-7.85	DRILLING FIRM / OPERATOR: RII / T.F.	DRILL RIG: CME-750X (SN 310218)	STATION / OFFSET: 496+53.86 / 201.8' Lt	EXPLORATION ID B-043-0-11
	TYPE: RETAINING WALL	SAMPLING FIRM / LOGGER: RII / S.M.	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED CL I-75	
	PID: 77889 BR ID: NA	DRILLING METHOD: 4.25" HSA	CALIBRATION DATE: 5/6/11	ELEVATION: 547.1 (MSL) EOB: 25.0 ft.	PAGE 1 OF 1
	START: 10/4/11 END: 10/4/11	SAMPLING METHOD: SPT	ENERGY RATIO (%): 77.1	LAT / LONG : 39.194406205 ° N / 84.47654299 ° W	

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N ₆₀	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%)					ATTERBERG			WC	ODOT CLASS (GI)	BACK FILL
								GR	CS	FS	SI	CL	LL	PL	PI			
0.7' - TOPSOIL (8.0")	547.1																	< > < >
FILL: MEDIUM DENSE, BROWN GRAVEL AND SAND, LITTLE SILT, TRACE CLAY, DRY. BRICK FRAGMENTS PRESENT IN SS-1		1	6															< > < >
		2	4	13	44	SS-1	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	< > < >
		3																< > < >
		4	5	15	44	SS-2	-	34	24	24	12	6	NP	NP	NP	6	A-1-b (0)	< > < >
STIFF, BROWN SILTY CLAY , SOME COARSE TO FINE SAND, TRACE FINE GRAVEL, DAMP TO MOIST.	541.6	5	7															< > < >
		6	4															< > < >
		7	5	12	56	SS-3	2.00	-	-	-	-	-	-	-	-	21	A-6b (V)	< > < >
		8																< > < >
	536.6	9	2	10	44	SS-4	1.50	10	13	16	35	26	36	19	17	17	A-6b (8)	< > < >
		10	6															< > < >
		11	17															< > < >
		12	23	62	78	SS-5	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	< > < >
MEDIUM DENSE TO VERY DENSE, BROWNISH GRAY GRAVEL AND SAND , TRACE SILT, TRACE CLAY, DAMP.		13	25															< > < >
		14	4	23	78	SS-6	-	-	-	-	-	-	-	-	-	4	A-1-b (V)	< > < >
		15	9															< > < >
		16	8															< > < >
	526.6	17	10	28	72	SS-7	-	24	48	20	7	1	NP	NP	NP	4	A-1-b (0)	< > < >
		18	12															< > < >
		19	9	39	78	SS-8	-	-	-	-	-	-	-	-	-	3	A-1-b (V)	< > < >
		20	13															< > < >
MEDIUM DENSE, BROWNISH GRAY FINE SAND , LITTLE COARSE SAND, LITTLE FINE GRAVEL, DRY.	522.1	21	13															< > < >
		22	13	28	83	SS-9	-	-	-	-	-	-	-	-	-	4	A-3 (V)	< > < >
		23	9															< > < >
		24	6	21	78	SS-10	-	-	-	-	-	-	-	-	-	4	A-3 (V)	< > < >
		25	7															< > < >

NOTES: GROUNDWATER NOT ENCOUNTERED DURING DRILLING; CAVE-IN DEPTH @ 3.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: COMPACTED WITH THE AUGER 25 LBS BENTONITE CHIPS AND SOIL CUTTINGS

APPENDIX V

LABORATORY TEST RESULTS



6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

CONSOLIDATED, UNDRAINED TRIAXIAL

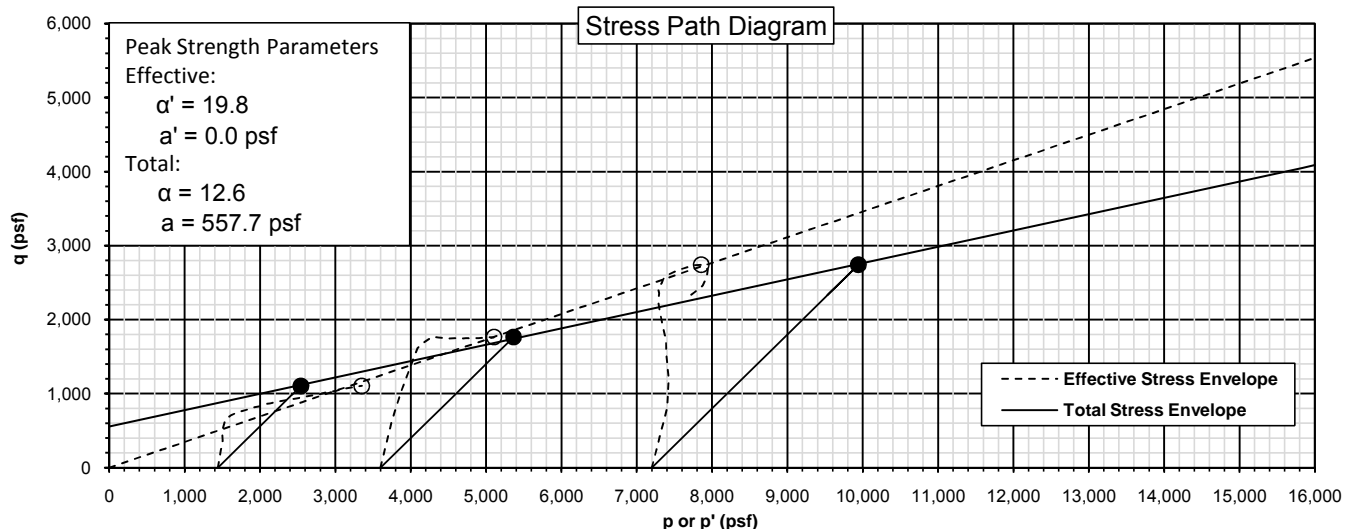
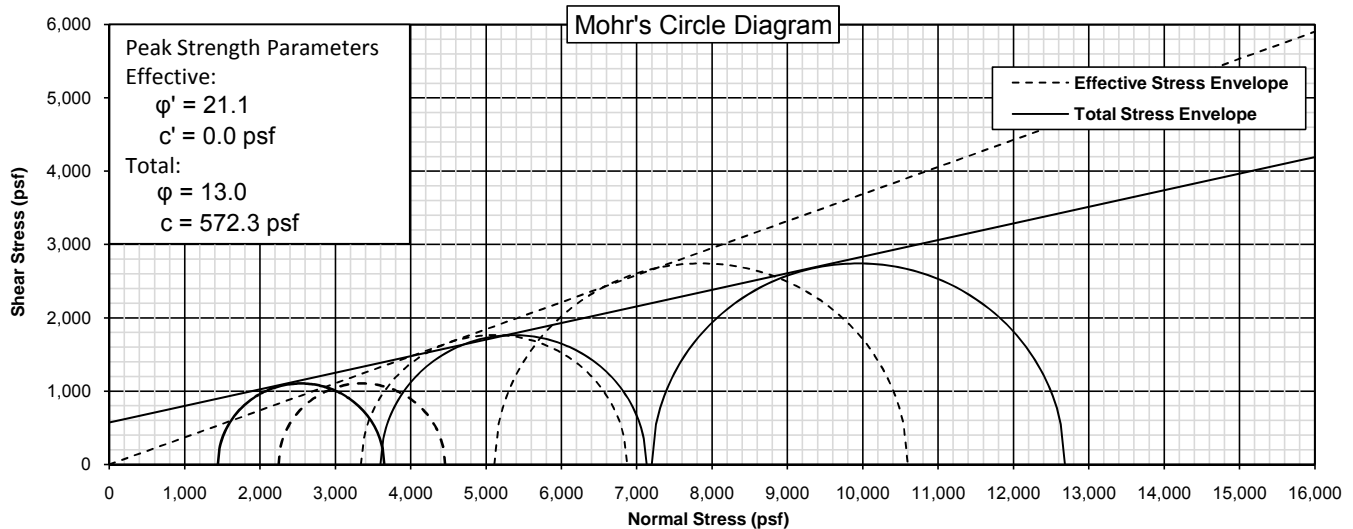
ASTM D-4767

PROJECT NAME: HAM-75-12.60
JOB NUMBER: B-10-024
BORING NUMBER: B-017-0-10
SAMPLE NUMBER: ST-8a
SAMPLE DEPTH: 19.0 ft - 20.5 ft
DATE OF TESTING: 9/20/2011 through 9/26/2011
TESTED BY: JH

Soil Description: CLAY With little silt trace sand and Gravel
Soil Classification: CL / A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	50	22	28	2.5	1.7	2.4	23.5	69.9

Stage	Boring No.	Sample No.	Depth (ft)	$(\sigma_3)_f$ (psf)	$(\sigma_1)_f$ (psf)	$(\sigma_3')_f$ (psf)	$(\sigma_1')_f$ (psf)	p'_f (psf)	q_f (psf)
1	B-017-0-10	ST-8a	19.0	7,200.0	12,684.3	5,112.0	10,596.3	7,854.2	2,742.2
2	B-017-0-10	ST-8b	19.5	3,600.0	7,130.6	3,340.8	6,871.4	5,106.1	1,765.3
3	B-017-0-10	ST-8c	20	1,440.0	3,648.6	2,246.4	4,455.0	3,350.7	1,104.3





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Fax Number: (614) 823-4990

CONSOLIDATED, UNDRAINED TRIAXIAL

ASTM D-4767

PROJECT NAME: HAM-75-12.60
JOB NUMBER: B-10-024
BORING NUMBER: B-017-0-10
SAMPLE NUMBER: ST-8a
SAMPLE DEPTH: 19.0 ft
DATE OF TESTING: 9/20/2011
TESTED BY: JH

Data for Specimen No. 1

Soil Description: CLAY With little silt trace sand and Gravel

Soil Classification: CL / A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	50	22	28	2.5	1.7	2.4	23.5	69.9

Diameter, D_0 : 2.869 in
Area, A_0 : 6.466 in²
Height, L_0 : 5.721 in
Volume, V_0 : 36.996 in³

Volume of Solids, V_s : 23.836 in³
Initial Volume of Voids, V_v : 13.160 in³
Initial Void Ratio, e_0 : 0.552
Initial Degree of Saturation, S_0 : 120.191 %

Water Content BEFORE Test

Tin No.: Paper g
Wet Soil + Tin : 197.89 g
Dry Soil + Tin : 164.53 g
Tin Weight : 30.3 g
Dry Mass : 134.23 g
Weight of water : 33.36 g
Moisture : 24.85 %

Water Content AFTER Test (Total Specimen)

Tin No.: KDW g
Wet Soil + Tin : 1331.20 g
Dry Soil + Tin : 1120.90 g
Tin Weight : 78.00 g
Dry Mass : 1042.90 g
Weight of water : 210.30 g
Moisture : 20.16 %
Wet Density : 129.05 pcf
Dry Density : 107.39 pcf

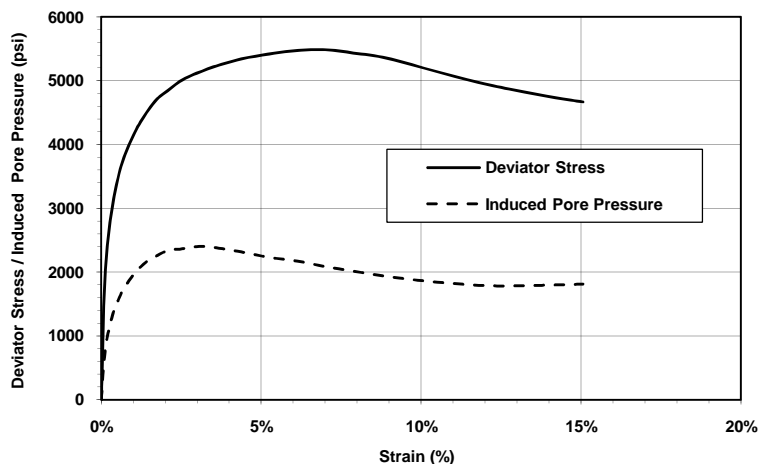
Consolidation Cell Pressure: 130.0 psi
Consolidation Back Pressure: 80.0 psi
Effective Confining Stress, σ_3 : 50.0 psi
7200.00 psf
Strain Rate: 0.0030 in/min

Deviator Stress @ Failure, D_s : 5484.32 psf
Axial Strain @ Failure: 7.01 %
Major Principal Stress @ Failure, σ_1 : 12684.32 psf
Induced Pore Pressure @ Failure: 2088.00 psf
Effective Minor Principal Stress, σ'_3 : 5112.00 psf
Effective Major Principal Stress, σ'_1 : 10596.32 psf

Failure Sketch



CU Compressive Strength and Induced Pore Pressure





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CONSOLIDATED, UNDRAINED TRIAXIAL

ASTM D-4767

PROJECT NAME: HAM-75-12.60
JOB NUMBER: B-10-024
BORING NUMBER: B-017-0-10
SAMPLE NUMBER: ST-8b
SAMPLE DEPTH: 19.5 ft
DATE OF TESTING: 9/22/2011
TESTED BY: JH

Data for Specimen No. 2

Soil Description: CLAY With little silt trace sand and Gravel

Soil Classification: CL / A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	50	22	28	2.5	1.7	2.4	23.5	69.9

Diameter, D_0 : 2.870 in
Area, A_0 : 6.469 in²
Height, L_0 : 5.867 in
Volume, V_0 : 37.957 in³

Volume of Solids, V_s : 23.427 in³
Initial Volume of Voids, V_v : 14.531 in³
Initial Void Ratio, e_0 : 0.620
Initial Degree of Saturation, S_0 : 106.98 %

Water Content BEFORE Test

Tin No.: Paper g
Wet Soil + Tin : 197.89 g
Dry Soil + Tin : 164.53 g
Tin Weight : 30.3 g
Dry Mass : 134.23 g
Weight of water : 33.36 g
Moisture : 24.85 %

Water Content AFTER Test (Total Specimen)

Tin No.: Freddy g
Wet Soil + Tin : 1346.50 g
Dry Soil + Tin : 1108.90 g
Tin Weight : 83.90 g
Dry Mass : 1025.00 g
Weight of water : 237.60 g
Moisture : 23.18 %
Wet Density : 126.72 pcf
Dry Density : 102.87 pcf

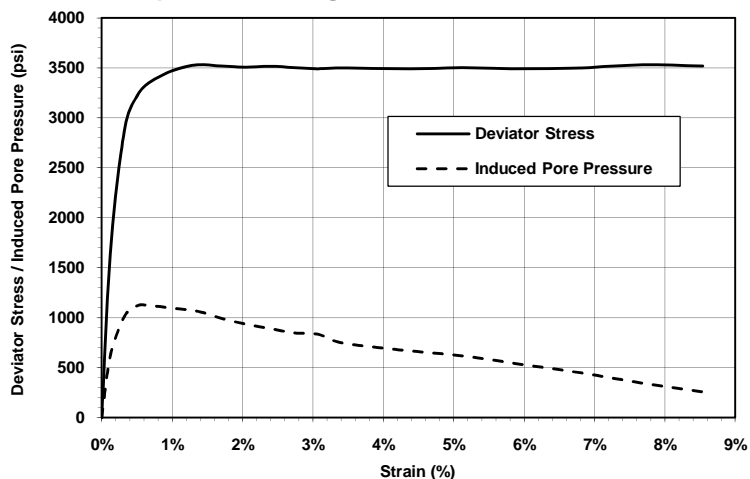
Consolidation Cell Pressure: 135.0 psi
Consolidation Back Pressure: 110.0 psi
Effective Confining Stress, σ_3 : 25.0 psi
3600.00 psf
Strain Rate: 0.0030 in/min

Deviator Stress @ Failure, D_s : 3530.56 psf
Axial Strain @ Failure: 8.54 %
Major Principal Stress @ Failure, σ_1 : 7130.56 psf
Induced Pore Pressure @ Failure: 259.20 psf
Effective Minor Principal Stress, σ'_3 : 3340.80 psf
Effective Major Principal Stress, σ'_1 : 6871.36 psf

Failure Sketch



CU Compressive Strength and Induced Pore Pressure





6350 Presidential Gateway
Columbus, Ohio 43231
Telephone: (614) 823-4949
Fax Number: (614) 823-4990

CONSOLIDATED, UNDRAINED TRIAXIAL

ASTM D-4767

PROJECT NAME: HAM-75-12.60
JOB NUMBER: B-10-024
BORING NUMBER: B-017-0-10
SAMPLE NUMBER: ST-8c
SAMPLE DEPTH: 20.0 ft
DATE OF TESTING: 9/26/2011
TESTED BY: Hoyt

Data for Specimen No. 3

Soil Description: CLAY With little silt trace sand and Gravel
Soil Classification: CL / A-7-6

Physical Characteristics	L.L.	P.L.	P.I.	Gravel%	C. Sand%	F. Sand%	Silt%	Clay%
	50	22	28	2.5	1.7	2.4	23.5	69.9

Diameter, D_0 : 2.869 in
Area, A_0 : 6.466 in²
Height, L_0 : 5.908 in
Volume, V_0 : 38.203 in³

Volume of Solids, V_s : 22.291 in³
Initial Volume of Voids, V_v : 15.912 in³
Initial Void Ratio, e_0 : 0.714
Initial Degree of Saturation, S_0 : 92.960 %

Water Content BEFORE Test

Tin No.: Paper g
Wet Soil + Tin : 197.89 g
Dry Soil + Tin : 164.53 g
Tin Weight : 30.3 g
Dry Mass : 134.23 g
Weight of water : 33.36 g
Moisture : 24.85286449 %

Water Content AFTER Test (Total Specimen)

Tin No.: KDW g
Wet Soil + Tin : 1334.20 g
Dry Soil + Tin : 1053.10 g
Tin Weight : 77.80 g
Dry Mass : 975.30 g
Weight of water : 281.10 g
Moisture : 28.82 %
Wet Density : 125.29 pcf
Dry Density : 97.26 pcf

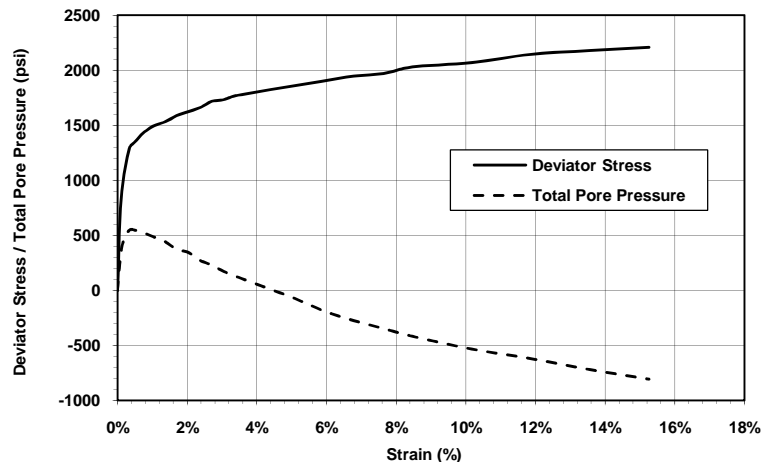
Consolidation Cell Pressure: 120.0 psi
Consolidation Back Pressure: 110.0 psi
Effective Confining Stress, σ_3 : 10.0 psi
1440.00 psf
Strain Rate: 0.0030 in/min

Deviator Stress @ Failure, D_s : 2208.58 psf
Axial Strain @ Failure: 15.25 %
Major Principal Stress @ Failure, σ_1 : 3648.58 psf
Induced Pore Pressure @ Failure: -806.40 psf
Effective Minor Principal Stress, σ_3' : 2246.40 psf
Effective Major Principal Stress, σ_1' : 4454.98 psf

Failure Sketch



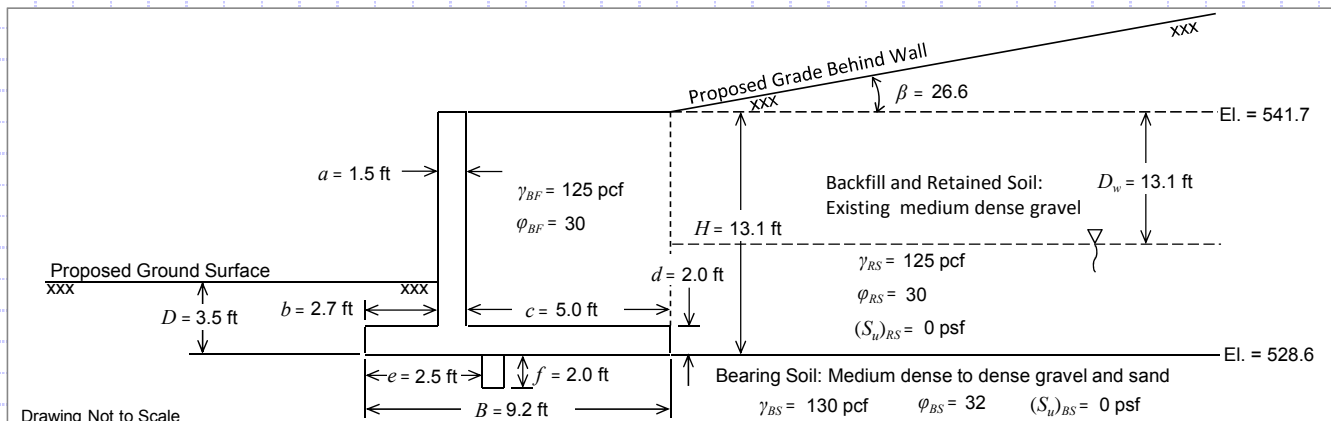
CU Compressive Strength and Induced Pore Pressure



APPENDIX VI

CIP WALL CALCULATIONS

Retaining Wall K - CIP Wall - 13.1 ft. Maximum Wall Height - B-040-0-11 through B-043-0-11



CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	26.6 °
Depth to Groundwater, (D_w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130 pcf
Bearing Soil Friction Angle, (ϕ_{BS}) =	32°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0 psf
Backfill and Retained Soil Unit Weight, (γ_{BF} , γ_{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (ϕ_{BF} , ϕ_{RS}) =	30°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0 psf
Active Earth Pressure Coefficient, (K_a) =	0.538
Passive Earth Pressure Coefficient, (K_p) =	3.255

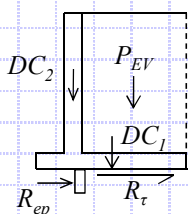
LRFD Load Factors

	DC	EV	EH	LS	EP	
Strength Ia	0.90	1.00	1.50	1.75	0.90	} (AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)
Strength Ib	1.25	1.35	1.50	1.75	0.90	
Service I	1.00	1.00	1.00	1.00	1.00	

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 10.6.3.4

Nominal Sliding Resisting:

$$R_n = R_\tau + R_{en}$$



$$R_\tau = P_v \tan \delta \quad \text{or} \quad R_\tau = \left((S_u)_{BS} \leq \frac{\sigma_v}{2} \right) \cdot B \quad (\text{Granular or Cohesive})$$

$$P_V = DC_1 + DC_2 + P_{EV} = \gamma_c \cdot [B \cdot d + (H - d) \cdot a] \cdot \gamma_{DC} + \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV}$$

$$\text{Granular} \left\{ \begin{array}{l} P_V = (150 \text{ pcf}) [(9.2 \text{ ft})(2.0 \text{ ft}) + (13.1 \text{ ft} - 2.0 \text{ ft})(1.5 \text{ ft})] (0.90) + \\ \quad (125 \text{ pcf})(13.1 \text{ ft} - 2.0 \text{ ft})(5.0 \text{ ft})(1.00) \end{array} \right. = 11.66925 \text{ kip/ft}$$

$$\tan \delta = \tan \varphi_{BS} = \tan(32) = 0.62$$

$$\sigma_v = \sigma_{DC_1} + \sigma_{DC_2} + \sigma_{EV} = \gamma_c \cdot [d + (H - d)] \cdot \gamma_{DC} + \gamma_{BF} \cdot (H - d) \cdot \gamma_{EV}$$

Cohesive $\sigma_y = N/A$

$$S_{11} = \text{N/A}$$

$$R_{\tau} = (11.66925 \text{ kip/ft})(0.62) = 7.23 \text{ kip/ft}$$

$$R_{ep} = \gamma_{BS} Df K_p \gamma_{ep} + \frac{1}{2} \gamma_{BS} f^2 K_p \gamma_{ep}$$

$$R_{en} = (130 \text{ pcf})(3.5 \text{ ft})(2.0 \text{ ft})(3.26)(0.90) + \frac{1}{2}(130 \text{ pcf})(2.0 \text{ ft})^2(3.26)(0.90) = 3.43 \text{ kip/ft}$$

$$R_u = 7.23 \text{ kip/ft} + 3.43 \text{ kip/ft} = 10.66 \text{ kip/ft}$$



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JOB HAM-75-7.85 NO. B-10-020
SHEET NO. 2 OF 8
CALCULATED BY BRT DATE 8/8/2013
CHECKED BY NCK DATE 8/8/2013
Retaining Wall K - 13.1 ft Maximum Wall Height

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D_w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130 pcf
Bearing Soil Friction Angle, (ϕ_{BS}) =	32°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0 psf
Backfill and Retained Soil Unit Weight, (γ_{BF} , γ_{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (ϕ_{BF} , ϕ_{RS}) =	30°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0 psf
Active Earth Pressure Coefficient, (K_a) =	0.538
Passive Earth Pressure Coefficient, (K_p) =	3.255

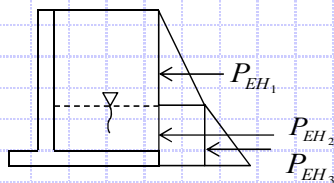
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Sliding (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 10.6.3.4 (Continued)

Sliding Force:



$$P_H = P_{EH_1} + P_{EH_2} + P_{EH_3}$$

$$P_{EH_1} = \frac{1}{2} \gamma_{RS} D_w^2 K_a \gamma_{EH}$$

$$P_{EH_1} = \frac{1}{2} (125 \text{ pcf}) (13.1 \text{ ft})^2 (0.538) (1.50) = 8.66 \text{ kip/ft}$$

$$P_{EH_2} = \gamma_{RS} D_w (H - D_w) K_a \gamma_{EH}$$

$$P_{EH_2} = (125 \text{ pcf}) (13.1 \text{ ft}) (13.1 \text{ ft} - 13.1 \text{ ft}) (0.538) (1.50) = 0.00 \text{ kip/ft}$$

$$P_{EH_3} = \frac{1}{2} \gamma'_{RS} (H - D_w)^2 K_a \gamma_{EH} + \frac{1}{2} \gamma_w (H - D_w)^2 \gamma_{WA}$$

$$P_{EH_3} = \frac{1}{2} (125 \text{ pcf}) (13.1 \text{ ft} - 13.1 \text{ ft})^2 (0.538) (1.50) + \frac{1}{2} (62.4 \text{ pcf}) (13.1 \text{ ft} - 13.1 \text{ ft})^2 (1.00) = 0.00 \text{ kip/ft}$$

$$P_H = 8.66 \text{ kip/ft} + 0.00 \text{ kip/ft} + 0.00 \text{ kip/ft}$$

$$P_H = 8.66 \text{ kip/ft}$$

Check Sliding Resistance

Use $\phi_\tau = 1.00$ Use $\phi_{ep} = 0.50$ (Per AASHTO LRFD BDM Tables 10.5.5.2.2-1 and 11.5.6-1)

$$P_H \leq \phi_n \cdot R_n \rightarrow P_H \leq \phi_\tau \cdot R_\tau + \phi_{ep} \cdot R_{ep}$$

$$8.66 \text{ kip/ft} \leq (7.23 \text{ kip/ft}) (1.00) + (3.43 \text{ kip/ft}) (0.50) = 8.95 \text{ kip/ft}$$

$$8.66 \text{ kip/ft} \leq 8.95 \text{ kip/ft}$$

OK



CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D _w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

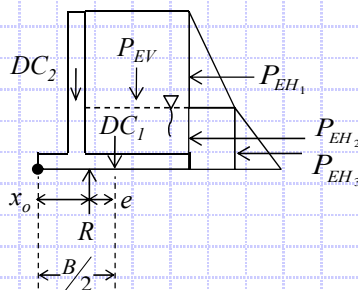
Bearing Soil Unit Weight, (γ _{BS}) =	130 pcf
Bearing Soil Friction Angle, (φ _{BS}) =	32°
Bearing Soil Undrained Shear Strength, [(s _u) _{BS}] =	0 psf
Backfill and Retained Soil Unit Weight, (γ _{BF} , γ _{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (φ _{BF} , φ _{RS}) =	30°
Retained Soil Undrained Shear Strength, [(s _u) _{RS}] =	0 psf
Active Earth Pressure Coefficient, (K _a) =	0.538
Passive Earth Pressure Coefficient, (K _p) =	3.255

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Eccentricity (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.6.3.3



$$e = B/2 - x_o$$

$$x_o = \frac{M_V - M_H}{P_V} = (65.66 \text{ kip-ft/ft} - 37.82 \text{ kip-ft/ft}) / (11.67 \text{ kip/ft}) = 2.39 \text{ ft}$$

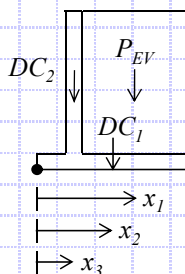
$$\begin{aligned} M_V &= 65.66 \text{ kip-ft/ft} \\ M_H &= 37.82 \text{ kip-ft/ft} \end{aligned} \quad \left. \begin{array}{l} \\ \end{array} \right\} \text{Defined below}$$

$$P_V = P_{EV} + DC_1 + DC_2 = 6.94 \text{ kip/ft} + 2.48 \text{ kip/ft} + 2.25 \text{ kip/ft} = 11.67 \text{ kip/ft}$$

$$e = (9.2 \text{ ft} / 2) - 2.39 \text{ ft} = 2.21 \text{ ft}$$

Resisting Moment, M_V :

$$M_V = P_{EV}(x_1) + DC_1(x_2) + DC_2(x_3)$$



$$P_{EV} = \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV} = (125 \text{ pcf})(13.1 \text{ ft} - 2.0 \text{ ft})(5.0 \text{ ft})(1.00) = 6.94 \text{ kip/ft}$$

$$DC_1 = \gamma_c \cdot B \cdot d \cdot \gamma_{DC} = (150 \text{ pcf})(9.2 \text{ ft})(2.0 \text{ ft})(0.90) = 2.48 \text{ kip/ft}$$

$$DC_2 = \gamma_c \cdot (H - d) \cdot a \cdot \gamma_{DC} = (150 \text{ pcf})(13.1 \text{ ft} - 2.0 \text{ ft})(1.5 \text{ ft})(0.90) = 2.25 \text{ kip/ft}$$

$$x_1 = a + b + c/2 = 1.5 \text{ ft} + 2.7 \text{ ft} + (5.0 \text{ ft} / 2) = 6.7 \text{ ft}$$

$$x_2 = B/2 = 9.2 \text{ ft} / 2 = 4.6 \text{ ft}$$

$$x_3 = b + a/2 = 2.7 \text{ ft} + (1.5 \text{ ft} / 2) = 3.5 \text{ ft}$$

$$M_V = (6.94 \text{ kip/ft})(6.7 \text{ ft}) + (2.48 \text{ kip/ft})(4.6 \text{ ft}) + (2.25 \text{ kip/ft})(3.5 \text{ ft}) = 65.66239 \text{ kip-ft/ft}$$



CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D _w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ _{BS}) =	130 pcf
Bearing Soil Friction Angle, (φ _{BS}) =	32°
Bearing Soil Undrained Shear Strength, [(s _u) _{BS}] =	0 psf
Backfill and Retained Soil Unit Weight, (γ _{BF} , γ _{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (φ _{BF} , φ _{RS}) =	30°
Retained Soil Undrained Shear Strength, [(s _u) _{RS}] =	0 psf
Active Earth Pressure Coefficient, (K _a) =	0.538
Passive Earth Pressure Coefficient, (K _p) =	3.255

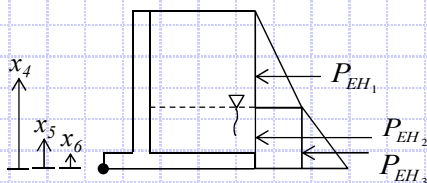
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Eccentricity (Loading Case - Strength Ia) - AASHTO LRFD BDM Section 11.6.3.3 (Continued)

Overturning Moment, M_H:



$$M_H = P_{EH1}(x_4) + P_{EH2}(x_5) + P_{EH3}(x_6)$$

$$P_{EH1} = \frac{1}{2} \gamma_{RS} D_w^2 K_a \gamma_{EH} = \frac{1}{2} (125 \text{ pcf}) (13.1 \text{ ft})^2 (0.538) (1.50) = 8.66 \text{ kip/ft}$$

$$P_{EH2} = \gamma_{RS} D_w (H - D_w) K_a \gamma_{EH} = (125 \text{ pcf}) (13.1 \text{ ft}) (13.1 \text{ ft} - 13.1 \text{ ft}) (0.538) (1.50) = 0.00 \text{ kip/ft}$$

$$P_{EH3} = \frac{1}{2} \gamma_{RS} (H - D_w)^2 K_a \gamma_{EH} + \frac{1}{2} \gamma_w (H - D_w)^2 \gamma_{WA}$$

$$P_{EH3} = \frac{1}{2} (125 \text{ pcf}) (13.1 \text{ ft} - 13.1 \text{ ft})^2 (0.538) (1.50) + \frac{1}{2} (62.4 \text{ pcf}) (13.1 \text{ ft} - 13.1 \text{ ft})^2 (1.00) = 0.00 \text{ kip/ft}$$

$$x_4 = (H - D_w) + \frac{D_w}{3} = (13.1 \text{ ft} - 13.1 \text{ ft}) + (13.1 \text{ ft} / 3) = 4.4 \text{ ft}$$

$$x_5 = \frac{(H - D_w)}{2} = (13.1 \text{ ft} - 13.1 \text{ ft}) / 2 = 0.0 \text{ ft}$$

$$x_6 = \frac{(H - D_w)}{3} = (13.1 \text{ ft} - 13.1 \text{ ft}) / 3 = 0.0 \text{ ft}$$

$$M_H = (8.66 \text{ kip/ft})(4.4 \text{ ft}) + (0.00 \text{ kip/ft})(0.0 \text{ ft}) + (0.00 \text{ kip/ft})(0.0 \text{ ft}) = 37.82 \text{ kip-ft/ft}$$

Limiting Eccentricity:

$$e_{\max} = \frac{B}{4} \rightarrow e_{\max} = (9.2 \text{ ft}) / 4 = 2.30 \text{ ft}$$

Check Eccentricity

$$e < e_{\max} \rightarrow 2.21 \text{ ft} < 2.30 \text{ ft} \quad \text{OK}$$



CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D_w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

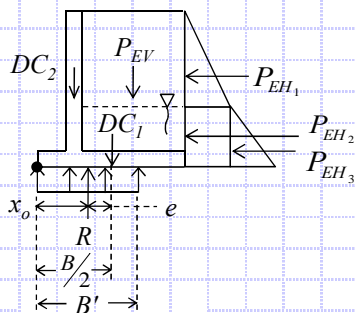
Bearing Soil Unit Weight, (γ_{BS}) =	130 pcf
Bearing Soil Friction Angle, (ϕ_{BS}) =	32°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0 psf
Backfill and Retained Soil Unit Weight, (γ_{BF}, γ_{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (ϕ_{BF}, ϕ_{RS}) =	30°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0 psf
Active Earth Pressure Coefficient, (K_a) =	0.538
Passive Earth Pressure Coefficient, (K_p) =	3.255

LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2



$$q_{eq} = \frac{P_V}{B'}$$

$$B' = B - 2e = 9.2 \text{ ft} - 2(1.36 \text{ ft}) = 6.48 \text{ ft}$$

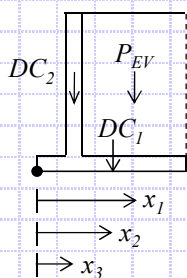
$$e = \frac{B}{2} - x_o = (9.2 \text{ ft} / 2) - 3.24 \text{ ft} = 1.36 \text{ ft}$$

$$x_o = \frac{M_V - M_H}{P_V} = (89.39 \text{ kip-ft/ft} - 37.82 \text{ kip-ft/ft}) / (15.94 \text{ kip/ft}) = 3.24 \text{ ft}$$

$$q_{eq} = (15.94 \text{ kip/ft}) / (6.48 \text{ ft}) = 2.46 \text{ ksf}$$

Resisting Moment, M_V :

$$M_V = P_{EV}(x_1) + DC_1(x_2) + DC_2(x_3)$$



$$P_{EV} = \gamma_{BF} \cdot (H - d) \cdot c \cdot \gamma_{EV} = (125 \text{ pcf})(13.1 \text{ ft} - 2.0 \text{ ft})(5.0 \text{ ft})(1.35) = 9.37 \text{ kip/ft}$$

$$DC_1 = \gamma_c \cdot B \cdot d \cdot \gamma_{DC} = (150 \text{ pcf})(9.2 \text{ ft})(2.0 \text{ ft})(1.25) = 3.45 \text{ kip/ft}$$

$$DC_2 = \gamma_c \cdot (H - d) \cdot a \cdot \gamma_{DC} = (150 \text{ pcf})(13.1 \text{ ft} - 2.0 \text{ ft})(1.5 \text{ ft})(1.25) = 3.12 \text{ kip/ft}$$

$$x_1 = a + b + \frac{c}{2} = 1.5 \text{ ft} + 2.7 \text{ ft} + (5.0 \text{ ft} / 2) = 6.7 \text{ ft}$$

$$x_2 = \frac{B}{2} = 9.2 \text{ ft} / 2 = 4.6 \text{ ft}$$

$$x_3 = b + \frac{a}{2} = 2.7 \text{ ft} + (1.5 \text{ ft} / 2) = 3.5 \text{ ft}$$

$$M_V = (9.37 \text{ kip/ft})(6.7 \text{ ft}) + (3.45 \text{ kip/ft})(4.6 \text{ ft}) + (3.12 \text{ kip/ft})(3.5 \text{ ft})$$

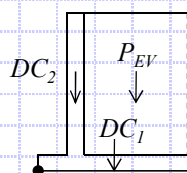
$$M_V = 89.39 \text{ kip-ft/ft}$$

Vertical Force, P_V :

$$P_V = P_{EV} + DC_1 + DC_2$$

$$P_V = 9.37 \text{ kip/ft} + 3.45 \text{ kip/ft} + 3.12 \text{ kip/ft}$$

$$P_V = 15.94 \text{ kip/ft}$$





CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D _w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ _{BS}) =	130 pcf
Bearing Soil Friction Angle, (φ _{BS}) =	32°
Bearing Soil Undrained Shear Strength, [(s _u) _{BS}] =	0 psf
Backfill and Retained Soil Unit Weight, (γ _{BF} , γ _{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (φ _{BF} , φ _{RS}) =	30°
Retained Soil Undrained Shear Strength, [(s _u) _{RS}] =	0 psf
Active Earth Pressure Coefficient, (K _a) =	0.538
Passive Earth Pressure Coefficient, (K _p) =	3.255

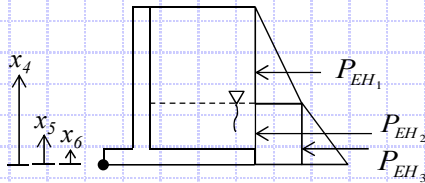
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

Check Bearing Capacity (Loading Case - Strength Ib) - AASHTO LRFD BDM Section 11.6.3.2 (Continued)

Overturning Moment, M_H:



$$M_H = P_{EH_1}(x_4) + P_{EH_2}(x_5) + P_{EH_3}(x_6)$$

$$P_{EH_1} = \frac{1}{2} \gamma_{RS} D_w^2 K_a \gamma_{EH} = \frac{1}{2} (125 \text{ pcf}) (13.1 \text{ ft})^2 (0.538) (1.50) = 8.66 \text{ kip/ft}$$

$$P_{EH_2} = \gamma_{RS} D_w (H - D_w) K_a \gamma_{EH} = (125 \text{ pcf}) (13.1 \text{ ft}) (13.1 \text{ ft} - 13.1 \text{ ft}) (0.538) (1.50) = 0.00 \text{ kip/ft}$$

$$P_{EH_3} = \frac{1}{2} \gamma_{RS} (H - D_w)^2 K_a \gamma_{EH} + \frac{1}{2} \gamma_w (H - D_w)^2 \gamma_{WA}$$

$$P_{EH_3} = \frac{1}{2} (125 \text{ pcf}) (13.1 \text{ ft} - 13.1 \text{ ft})^2 (0.538) (1.50) + \frac{1}{2} (62.4 \text{ pcf}) (13.1 \text{ ft} - 13.1 \text{ ft})^2 (1.00) = 0.00 \text{ kip/ft}$$

$$x_4 = (H - D_w) + \frac{D_w}{3} = (13.1 \text{ ft} - 13.1 \text{ ft}) + (13.1 \text{ ft} / 3) = 4.4 \text{ ft}$$

$$x_5 = \frac{(H - D_w)}{2} = (13.1 \text{ ft} - 13.1 \text{ ft}) / 2 = 0.0 \text{ ft}$$

$$x_6 = \frac{(H - D_w)}{3} = (13.1 \text{ ft} - 13.1 \text{ ft}) / 3 = 0.0 \text{ ft}$$

$$M_H = (8.66 \text{ kip/ft})(4.4 \text{ ft}) + (0.00 \text{ kip/ft})(0.0 \text{ ft}) + (0.00 \text{ kip/ft})(0.0 \text{ ft}) = 37.82 \text{ kip-ft/ft}$$

Nominal Bearing Resistance:

$$N_c = 44.0$$

$$N_q = 28.5$$

$$N_\gamma = 28.0$$

$$q_n = cN_c + \gamma DN_q + \frac{1}{2} \gamma B N_\gamma = (0 \text{ psf})(44) + (130 \text{ pcf})(3.5 \text{ ft})(28.5) + \frac{1}{2} (130 \text{ pcf})(9.2 \text{ ft})(28.0) = 29.75 \text{ ksf}$$

Check Bearing Capacity

$$\text{Use } \phi_b = 0.55 \text{ (Per AASHTO LRFD BDM Table 11.5.6-1)}$$

$$q_{eq} \leq q_n \cdot \phi_b \rightarrow 2.46 \text{ ksf} \leq (29.7 \text{ ksf})(0.55) = 16.36 \text{ ksf} \rightarrow 2.46 \text{ ksf} \leq 16.36 \text{ ksf}$$

OK



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JOB HAM-75-7.85 NO. B-10-020
SHEET NO. 7 OF 8
CALCULATED BY BRT DATE 8/8/2013
CHECKED BY NCK DATE 8/8/2013
Retaining Wall K - 13.1 ft Maximum Wall Height

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D_w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130 pcf
Bearing Soil Friction Angle, (ϕ_{BS}) =	32°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0 psf
Backfill and Retained Soil Unit Weight, (γ_{BF} , γ_{RS}) =	125 pcf
Backfill and Retained Soil Friction Angle, (ϕ_{BF} , ϕ_{RS}) =	30°
Retained Soil Undrained Shear Strength, [$(s_u)_{RS}$] =	0 psf
Active Earth Pressure Coefficient, (K_a) =	0.538
Passive Earth Pressure Coefficient, (K_p) =	3.255

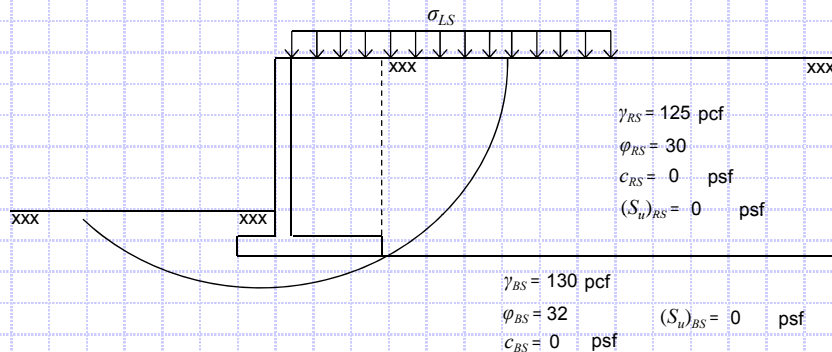
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

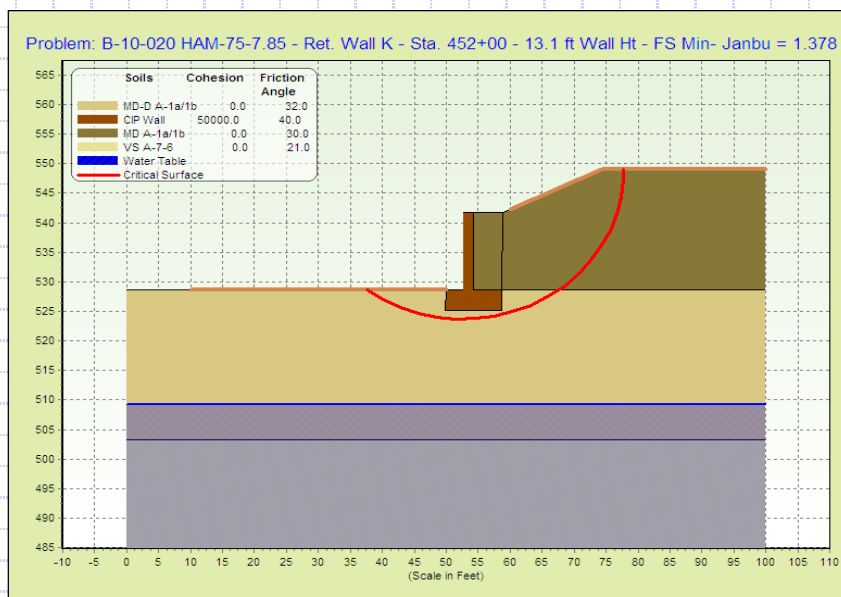
Check Overall (Global) Stability (Loading Case - Service I) - AASHTO LRFD BDM Sections 11.6.2.3 and 11.10.5.2

Long Term Stability - Drained Conditions



Loading scenario modeled as shown to the left and analyzed for slope stability using STABL for Windows software.

Graphical output shown below and tabular output results are provided as a separate attachment.



Check Overall (Global) Stability

$$1.0 \leq FS \cdot \phi_{GS} \rightarrow 1.0 \leq (1.378)(0.75) = 1.03 \rightarrow 1.0 \leq 1.03 \quad \text{OK}$$

FS = **1.378** (From STABL Slope Stability Analysis)

Use ϕ_{GS} = **0.75** (Per AASHTO LRFD BDM Section 11.6.2.3)



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JOB HAM-75-7.85 NO. B-10-020
SHEET NO. 8 OF 8
CALCULATED BY BRT DATE 8/8/2013
CHECKED BY NCK DATE 8/8/2013
Retaining Wall K - 13.1 ft Maximum Wall Height

CIP Wall Dimensions and Surcharge Loading

Wall Height, (H) =	13.1 ft
Foundation Width (Entire Base Width), (B) =	9.2 ft
Stem Width, (a) =	1.5 ft
Toe Width, (b) =	2.7 ft
Heel Width, (c) =	5.0 ft
Footing Thickness, (d) =	2.0 ft
Location of Shear Key, (e) =	2.5 ft
Depth of Shear Key, (f) =	2.0 ft
Embedment Depth, (D) =	3.5 ft
Retained Soil Backslope, (β) =	27°
Depth to Groundwater, (D_w) =	13.1 ft

Bearing and Retained/Backfill Soil Properties:

Bearing Soil Unit Weight, (γ_{BS}) =	130 pcf
Bearing Soil Friction Angle, (ϕ_{BS}) =	32°
Bearing Soil Undrained Shear Strength, [$(s_u)_{BS}$] =	0 psf
Backfill and Retained Soil Unit Weight, (γ_{BF} , γ_{RS}) =	125 pcf
Retained Soil Friction Angle, (ϕ_{RS}) =	30°
Backfill and Retained Soil Friction Angle, (ϕ_{BF} , ϕ_{RS}) =	0 psf
Active Earth Pressure Coefficient, (K_a) =	0.538
Passive Earth Pressure Coefficient, (K_p) =	3.255

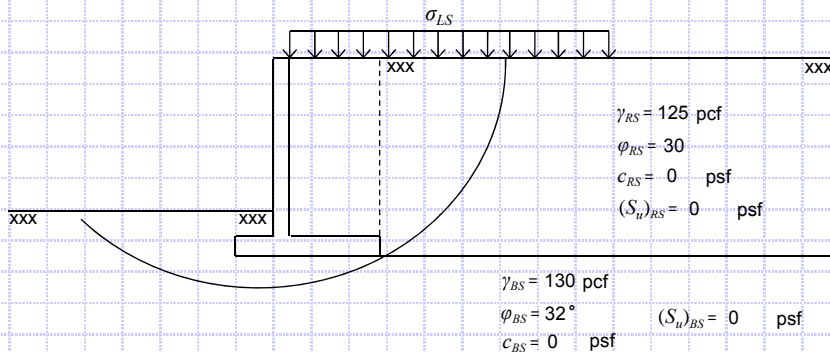
LRFD Load Factors

	DC	EV	EH	LS	EP
Strength Ia	0.90	1.00	1.50	1.75	0.90
Strength Ib	1.25	1.35	1.50	1.75	0.90
Service I	1.00	1.00	1.00	1.00	1.00

(AASHTO LRFD BDM Tables 3.4.1-1 and 3.4.1-2 - Active Earth Pressure)

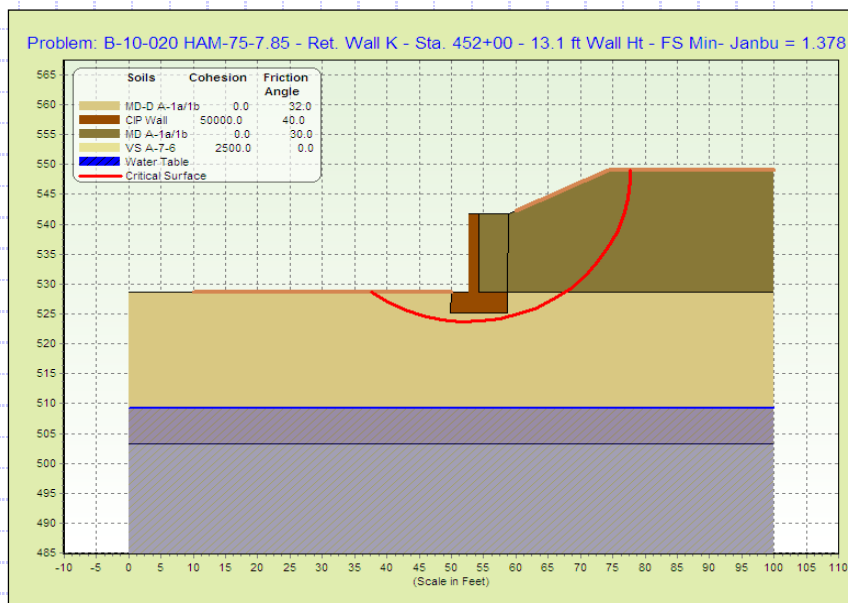
Check Overall (Global) Stability (Loading Case - Service I) - AASHTO LRFD BDM Sections 11.6.2.3 and 11.10.5.2 (Cont'd)

Short Term Stability - Undrained Conditions



Loading scenario modeled as shown to the left and analyzed for slope stability using STABL for Windows software.

Graphical output shown below and tabular output results are provided as a separate attachment.



Check Overall (Global) Stability

$$1.0 \leq FS \cdot \phi_{GS} \rightarrow 1.0 \leq (1.378)(0.75) = 1.03 \rightarrow 1.0 \leq 1.03 \quad \text{OK}$$

FS = **1.378** (From STABL Slope Stability Analysis)

Use ϕ_{GS} = **0.75** (Per AASHTO LRFD BDM Section 11.6.2.3)

result.out

** STABL for WINDOWS **
by
Geotechnical Software Solutions

1

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:
Time of Run:
Run By:
Input Data Filename: run.in
Output Filename: result.out
Unit: U.S.C.
Plotted Output Filename: result.plt

PROBLEM DESCRIPTION B-10-020 HAM-75-7.85 - Ret. Wall K - Sta
. 452+00 - 13.1 ft Wall Ht

BOUNDARY COORDINATES

7 Top Boundaries
15 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	528.60	50.00	528.60	2
2	50.00	528.60	52.70	528.60	4
3	52.70	528.60	52.80	541.70	4
4	52.80	541.70	54.30	541.70	4
5	54.30	541.70	58.80	541.70	1
6	58.80	541.70	74.30	549.00	1
7	74.30	549.00	100.00	549.00	1
8	54.20	528.60	54.30	541.70	1
9	54.20	528.60	58.70	528.60	4
10	58.70	528.60	58.80	541.70	1
11	58.70	528.60	100.00	528.60	2
12	49.90	525.10	50.00	528.60	4
13	49.90	525.10	58.60	525.10	2
14	58.60	525.10	58.70	528.60	2
15	0.00	503.30	100.00	503.30	3

1

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	0.0	30.0	0.00	0.0	1
2	130.0	135.0	0.0	32.0	0.00	0.0	1
3	120.0	130.0	0.0	21.0	0.00	0.0	1
4	150.0	150.0	50000.0	40.0	0.00	0.0	0

1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points
Page 1

result.out

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	509.30
2	100.00	509.30

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

750 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 75 Points Equally Spaced Along The Ground Surface Between X = 10.00 ft.
and X = 50.00 ft.

Each Surface Terminates Between X = 60.00 ft.
and X = 100.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 400.00 ft.

3.00 ft. Line Segments Define Each Trial Failure Surface.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	37.57	528.60
2	40.09	526.97
3	42.78	525.65
4	45.61	524.65
5	48.54	524.00
6	51.52	523.70
7	54.52	523.76
8	57.49	524.17
9	60.40	524.93
10	63.19	526.03
11	65.83	527.45
12	68.28	529.17
13	70.52	531.18
14	72.50	533.43
15	74.19	535.90
16	75.59	538.56
17	76.66	541.36
18	77.39	544.27
19	77.77	547.25
20	77.78	549.00

*** 1.378 ***

Individual data on the 30 slices

result.out

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	2.5	267.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	2.7	802.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	2.8	1270.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	2.9	1627.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	1.4	827.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	0.1	65.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	1.5	1060.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	1.2	830.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	0.1	168.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	1.4	3734.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.1	250.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.2	518.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	3.0	6865.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	1.1	2506.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.1	220.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.1	217.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	1.6	3492.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	2.8	6195.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	2.6	5852.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	1.6	3561.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.8	1743.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	2.2	4609.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	2.0	3805.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	1.7	2947.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.1	171.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
26	1.3	1878.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	1.1	1207.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.7	563.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.4	153.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.0	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	31.62	528.60
2	34.23	527.11
3	36.96	525.87
4	39.79	524.89
5	42.71	524.17
6	45.67	523.73
7	48.67	523.55
8	51.67	523.66
9	54.64	524.04
10	57.57	524.70
11	60.43	525.62
12	63.18	526.80
13	65.82	528.23
14	68.32	529.89
15	70.65	531.78
16	72.79	533.88
17	74.74	536.16
18	76.46	538.62
19	77.95	541.22
20	79.20	543.95
21	80.19	546.78
22	80.74	549.00

*** 1.384 ***

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.84	528.60

result.out

2	30.31	526.89
3	32.93	525.44
4	35.69	524.25
5	38.54	523.35
6	41.48	522.73
7	44.46	522.40
8	47.46	522.37
9	50.45	522.64
10	53.40	523.21
11	56.27	524.06
12	59.05	525.20
13	61.70	526.61
14	64.20	528.27
15	66.52	530.17
16	68.64	532.29
17	70.54	534.61
18	72.20	537.11
19	73.60	539.76
20	74.74	542.54
21	75.59	545.41
22	76.16	548.36
23	76.21	549.00

*** 1.422 ***

Failure Surface Specified By 22 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	35.41	528.60
2	37.99	527.07
3	40.71	525.81
4	43.54	524.81
5	46.45	524.10
6	49.42	523.68
7	52.42	523.56
8	55.42	523.73
9	58.38	524.19
10	61.28	524.95
11	64.10	525.98
12	66.80	527.29
13	69.36	528.85
14	71.75	530.66
15	73.96	532.70
16	75.95	534.94
17	77.71	537.37
18	79.23	539.96
19	80.48	542.68
20	81.46	545.52
21	82.16	548.44
22	82.23	549.00

*** 1.433 ***

1

Failure Surface Specified By 24 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	25.14	528.60
2	27.67	527.00
3	30.33	525.61
4	33.10	524.46
5	35.96	523.54
6	38.88	522.87
7	41.85	522.45
8	44.85	522.27

result.out

9	47.85	522.36
10	50.83	522.69
11	53.77	523.27
12	56.66	524.10
13	59.46	525.17
14	62.16	526.47
15	64.75	527.99
16	67.19	529.72
17	69.49	531.66
18	71.61	533.78
19	73.54	536.07
20	75.28	538.52
21	76.80	541.10
22	78.10	543.81
23	79.17	546.61
24	79.86	549.00

*** 1.477 ***

Failure Surface Specified By 24 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	29.46	528.60
2	31.98	526.97
3	34.63	525.57
4	37.40	524.41
5	40.26	523.50
6	43.19	522.86
7	46.16	522.47
8	49.16	522.35
9	52.16	522.50
10	55.13	522.92
11	58.05	523.60
12	60.90	524.53
13	63.66	525.72
14	66.29	527.15
15	68.80	528.80
16	71.14	530.68
17	73.31	532.75
18	75.28	535.01
19	77.05	537.44
20	78.59	540.01
21	79.89	542.71
22	80.95	545.52
23	81.76	548.41
24	81.87	549.00

*** 1.485 ***

1

Failure Surface Specified By 25 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	26.22	528.60
2	28.87	527.19
3	31.61	525.99
4	34.44	524.98
5	37.33	524.19
6	40.28	523.62
7	43.26	523.26
8	46.25	523.12
9	49.25	523.21
10	52.24	523.51
11	55.19	524.04
12	58.10	524.78

13	60.94	525.73	result.out
14	63.71	526.89	
15	66.38	528.25	
16	68.95	529.81	
17	71.39	531.55	
18	73.70	533.46	
19	75.87	535.54	
20	77.87	537.78	
21	79.70	540.15	
22	81.35	542.65	
23	82.82	545.27	
24	84.09	547.99	
25	84.47	549.00	

*** 1.490 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.30	528.60
2	29.72	526.83
3	32.30	525.30
4	35.01	524.01
5	37.83	522.99
6	40.74	522.24
7	43.70	521.77
8	46.69	521.58
9	49.69	521.67
10	52.67	522.05
11	55.60	522.70
12	58.45	523.63
13	61.20	524.83
14	63.83	526.27
15	66.31	527.96
16	68.61	529.88
17	70.73	532.01
18	72.64	534.32
19	74.32	536.81
20	75.75	539.44
21	76.94	542.20
22	77.85	545.06
23	78.50	547.99
24	78.62	549.00

*** 1.500 ***

1

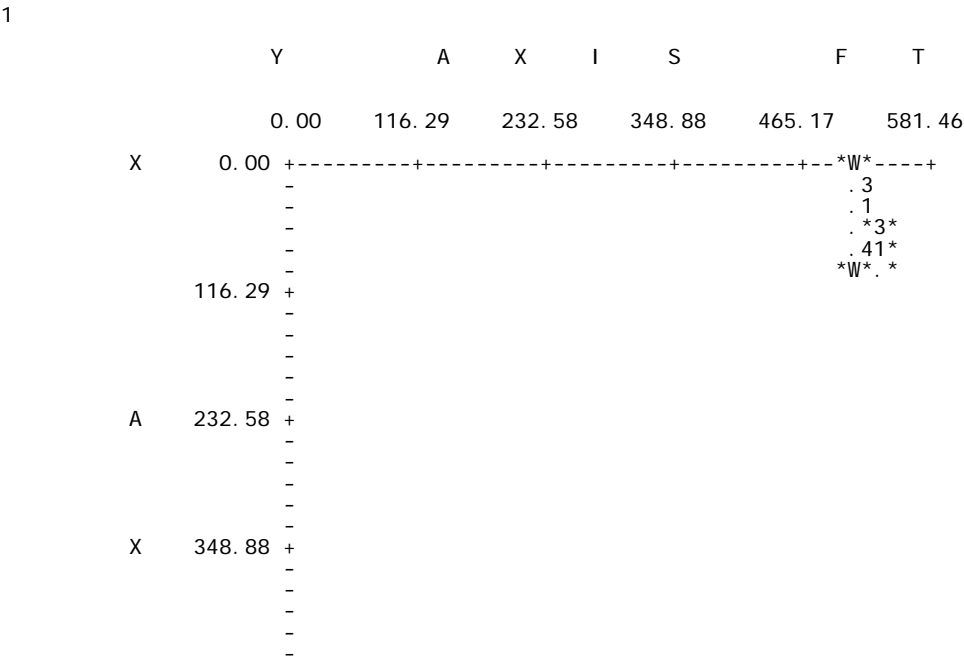
Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	36.49	528.60
2	39.13	527.18
3	41.89	526.00
4	44.74	525.08
5	47.67	524.41
6	50.64	524.02
7	53.64	523.89
8	56.64	524.03
9	59.61	524.44
10	62.53	525.12
11	65.38	526.06
12	68.13	527.25
13	70.77	528.68
14	73.26	530.35
15	75.60	532.23

			result. out
16	77.75	534.32	
17	79.71	536.59	
18	81.46	539.03	
19	82.98	541.62	
20	84.26	544.33	
21	85.29	547.15	
22	85.78	549.00	
*** 1.507 ***			

Failure Surface Specified By 22 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	37.57	528.60
2	40.23	527.21
3	43.00	526.07
4	45.87	525.18
5	48.80	524.55
6	51.78	524.20
7	54.78	524.12
8	57.77	524.30
9	60.74	524.76
10	63.65	525.49
11	66.48	526.47
12	69.21	527.71
13	71.82	529.19
14	74.29	530.90
15	76.58	532.83
16	78.70	534.96
17	80.61	537.27
18	82.31	539.75
19	83.77	542.37
20	84.99	545.11
21	85.96	547.95
22	86.22	549.00
*** 1.517 ***		



I	465.17	+	
		-	
		-	
		-	
		-	
S	581.46	+	
		-	
		-	
		-	
		-	
	697.75	+	
		-	
		-	
		-	
		-	
F	814.04	+	
		-	
		-	
		-	
		-	
T	930.34	+	

result.out

result.out

** STABL for WINDOWS **
by
Geotechnical Software Solutions

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer's Method of Slices

Run Date:
Time of Run:
Run By:
Input Data Filename: run.in
Output Filename: result.out
Unit: U.S.C.
Plotted Output Filename: result.plt

PROBLEM DESCRIPTION B-10-020 HAM-75-7.85 - Ret. Wall K - Sta
. 452+00 - 13.1 ft Wall Ht

BOUNDARY COORDINATES

7 Top Boundaries
15 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	528.60	50.00	528.60	2
2	50.00	528.60	52.70	528.60	4
3	52.70	528.60	52.80	541.70	4
4	52.80	541.70	54.30	541.70	4
5	54.30	541.70	58.80	541.70	1
6	58.80	541.70	74.30	549.00	1
7	74.30	549.00	100.00	549.00	1
8	54.20	528.60	54.30	541.70	1
9	54.20	528.60	58.70	528.60	4
10	58.70	528.60	58.80	541.70	1
11	58.70	528.60	100.00	528.60	2
12	49.90	525.10	50.00	528.60	4
13	49.90	525.10	58.60	525.10	2
14	58.60	525.10	58.70	528.60	2
15	0.00	503.30	100.00	503.30	3

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	0.0	30.0	0.00	0.0	1
2	130.0	135.0	0.0	32.0	0.00	0.0	1
3	120.0	130.0	2500.0	0.0	0.00	0.0	1
4	150.0	150.0	50000.0	40.0	0.00	0.0	0

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 2 Coordinate Points
Page 1

result.out

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	509.30
2	100.00	509.30

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

750 Trial Surfaces Have Been Generated.

10 Surfaces Initiate From Each Of 75 Points Equally Spaced Along The Ground Surface Between X = 10.00 ft.
and X = 50.00 ft.

Each Surface Terminates Between X = 60.00 ft.
and X = 100.00 ft.

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 400.00 ft.

3.00 ft. Line Segments Define Each Trial Failure Surface.

1

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Examined. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Janbu Method * *

Failure Surface Specified By 20 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	37.57	528.60
2	40.09	526.97
3	42.78	525.65
4	45.61	524.65
5	48.54	524.00
6	51.52	523.70
7	54.52	523.76
8	57.49	524.17
9	60.40	524.93
10	63.19	526.03
11	65.83	527.45
12	68.28	529.17
13	70.52	531.18
14	72.50	533.43
15	74.19	535.90
16	75.59	538.56
17	76.66	541.36
18	77.39	544.27
19	77.77	547.25
20	77.78	549.00

*** 1.378 ***

Individual data on the 30 slices

result.out

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Force Norm (lbs)	Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	2.5	267.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	2.7	802.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	2.8	1270.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	2.9	1627.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	1.4	827.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	0.1	65.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	1.5	1060.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	1.2	830.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	0.1	168.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	1.4	3734.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.1	250.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.2	518.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	3.0	6865.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	1.1	2506.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.1	220.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.1	217.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	1.6	3492.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	2.8	6195.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	2.6	5852.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	1.6	3561.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.8	1743.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	2.2	4609.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	2.0	3805.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	1.7	2947.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.1	171.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
26	1.3	1878.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	1.1	1207.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.7	563.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.4	153.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.0	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	31.62	528.60
2	34.23	527.11
3	36.96	525.87
4	39.79	524.89
5	42.71	524.17
6	45.67	523.73
7	48.67	523.55
8	51.67	523.66
9	54.64	524.04
10	57.57	524.70
11	60.43	525.62
12	63.18	526.80
13	65.82	528.23
14	68.32	529.89
15	70.65	531.78
16	72.79	533.88
17	74.74	536.16
18	76.46	538.62
19	77.95	541.22
20	79.20	543.95
21	80.19	546.78
22	80.74	549.00

*** 1.384 ***

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.84	528.60

result.out

2	30.31	526.89
3	32.93	525.44
4	35.69	524.25
5	38.54	523.35
6	41.48	522.73
7	44.46	522.40
8	47.46	522.37
9	50.45	522.64
10	53.40	523.21
11	56.27	524.06
12	59.05	525.20
13	61.70	526.61
14	64.20	528.27
15	66.52	530.17
16	68.64	532.29
17	70.54	534.61
18	72.20	537.11
19	73.60	539.76
20	74.74	542.54
21	75.59	545.41
22	76.16	548.36
23	76.21	549.00

*** 1.422 ***

Failure Surface Specified By 22 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	35.41	528.60
2	37.99	527.07
3	40.71	525.81
4	43.54	524.81
5	46.45	524.10
6	49.42	523.68
7	52.42	523.56
8	55.42	523.73
9	58.38	524.19
10	61.28	524.95
11	64.10	525.98
12	66.80	527.29
13	69.36	528.85
14	71.75	530.66
15	73.96	532.70
16	75.95	534.94
17	77.71	537.37
18	79.23	539.96
19	80.48	542.68
20	81.46	545.52
21	82.16	548.44
22	82.23	549.00

*** 1.433 ***

1

Failure Surface Specified By 24 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	25.14	528.60
2	27.67	527.00
3	30.33	525.61
4	33.10	524.46
5	35.96	523.54
6	38.88	522.87
7	41.85	522.45
8	44.85	522.27

result.out

9	47.85	522.36
10	50.83	522.69
11	53.77	523.27
12	56.66	524.10
13	59.46	525.17
14	62.16	526.47
15	64.75	527.99
16	67.19	529.72
17	69.49	531.66
18	71.61	533.78
19	73.54	536.07
20	75.28	538.52
21	76.80	541.10
22	78.10	543.81
23	79.17	546.61
24	79.86	549.00

*** 1.477 ***

Failure Surface Specified By 24 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	29.46	528.60
2	31.98	526.97
3	34.63	525.57
4	37.40	524.41
5	40.26	523.50
6	43.19	522.86
7	46.16	522.47
8	49.16	522.35
9	52.16	522.50
10	55.13	522.92
11	58.05	523.60
12	60.90	524.53
13	63.66	525.72
14	66.29	527.15
15	68.80	528.80
16	71.14	530.68
17	73.31	532.75
18	75.28	535.01
19	77.05	537.44
20	78.59	540.01
21	79.89	542.71
22	80.95	545.52
23	81.76	548.41
24	81.87	549.00

*** 1.485 ***

1

Failure Surface Specified By 25 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	26.22	528.60
2	28.87	527.19
3	31.61	525.99
4	34.44	524.98
5	37.33	524.19
6	40.28	523.62
7	43.26	523.26
8	46.25	523.12
9	49.25	523.21
10	52.24	523.51
11	55.19	524.04
12	58.10	524.78

13	60.94	525.73	result.out
14	63.71	526.89	
15	66.38	528.25	
16	68.95	529.81	
17	71.39	531.55	
18	73.70	533.46	
19	75.87	535.54	
20	77.87	537.78	
21	79.70	540.15	
22	81.35	542.65	
23	82.82	545.27	
24	84.09	547.99	
25	84.47	549.00	

*** 1.490 ***

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	27.30	528.60
2	29.72	526.83
3	32.30	525.30
4	35.01	524.01
5	37.83	522.99
6	40.74	522.24
7	43.70	521.77
8	46.69	521.58
9	49.69	521.67
10	52.67	522.05
11	55.60	522.70
12	58.45	523.63
13	61.20	524.83
14	63.83	526.27
15	66.31	527.96
16	68.61	529.88
17	70.73	532.01
18	72.64	534.32
19	74.32	536.81
20	75.75	539.44
21	76.94	542.20
22	77.85	545.06
23	78.50	547.99
24	78.62	549.00

*** 1.500 ***

1

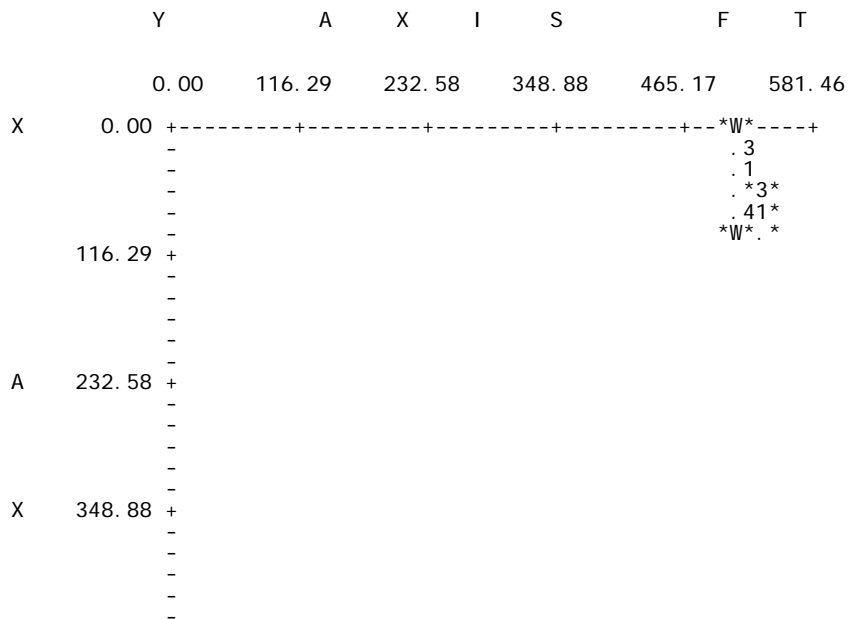
Failure Surface Specified By 22 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	36.49	528.60
2	39.13	527.18
3	41.89	526.00
4	44.74	525.08
5	47.67	524.41
6	50.64	524.02
7	53.64	523.89
8	56.64	524.03
9	59.61	524.44
10	62.53	525.12
11	65.38	526.06
12	68.13	527.25
13	70.77	528.68
14	73.26	530.35
15	75.60	532.23

16	77.75	534.32
17	79.71	536.59
18	81.46	539.03
19	82.98	541.62
20	84.26	544.33
21	85.29	547.15
22	85.78	549.00

Failure Surface Specified By 22 Coordinate Points

Poi nt No.	X-Surf (ft)	Y-Surf (ft)
1	37. 57	528. 60
2	40. 23	527. 21
3	43. 00	526. 07
4	45. 87	525. 18
5	48. 80	524. 55
6	51. 78	524. 20
7	54. 78	524. 12
8	57. 77	524. 30
9	60. 74	524. 76
10	63. 65	525. 49
11	66. 48	526. 47
12	69. 21	527. 71
13	71. 82	529. 19
14	74. 29	530. 90
15	76. 58	532. 83
16	78. 70	534. 96
17	80. 61	537. 27
18	82. 31	539. 75
19	83. 77	542. 37
20	84. 99	545. 11
21	85. 96	547. 95
22	86. 22	549. 00



		result	t.out
I	465.17	+	
		-	
		-	
		-	
		-	
S	581.46	+	
		-	
		-	
		-	
		-	
	697.75	+	
		-	
		-	
		-	
		-	
F	814.04	+	
		-	
		-	
		-	
		-	
T	930.34	+	